US Army Corps of Engineers
Waterways Experiment Station

Technical Report HL-95-13
October 1995

Redeye Crossing Reach, Lower Mississippi River

Report 1
Sediment Investigation

by T. J. Pokrefke, Jr., C. R. Nickles, N. K. Raphelt, M. J. Trawle, M. B. Boyd

SELECTE
DEC 2 0 1995
F

Approved For Public Release; Distribution Is Unlimited

19951218 080

"Original contains color plates: All DTIC reproductions will be in black and white"

DTIC QUALITY INSPECTED 1

Prepared for U.S. Army Engineer District, New Orleans

DISCLAIMER NOTICE

THIS DOCUMENT IS BEST QUALITY AVAILABLE. THE COPY FURNISHED TO DTIC CONTAINED A SIGNIFICANT NUMBER OF COLOR PAGES WHICH DO NOT REPRODUCE LEGIBLY ON BLACK AND WHITE MICROFICHE.

Technical Report HL-95-13
October 1995

Redeye Crossing Reach, Lower Mississippi River

Report 1
Sediment Investigation

by T. J. Pokrefke, Jr., C. R. Nickles, N. K. Raphelt,
M. J. Trawle, M. B. Boyd

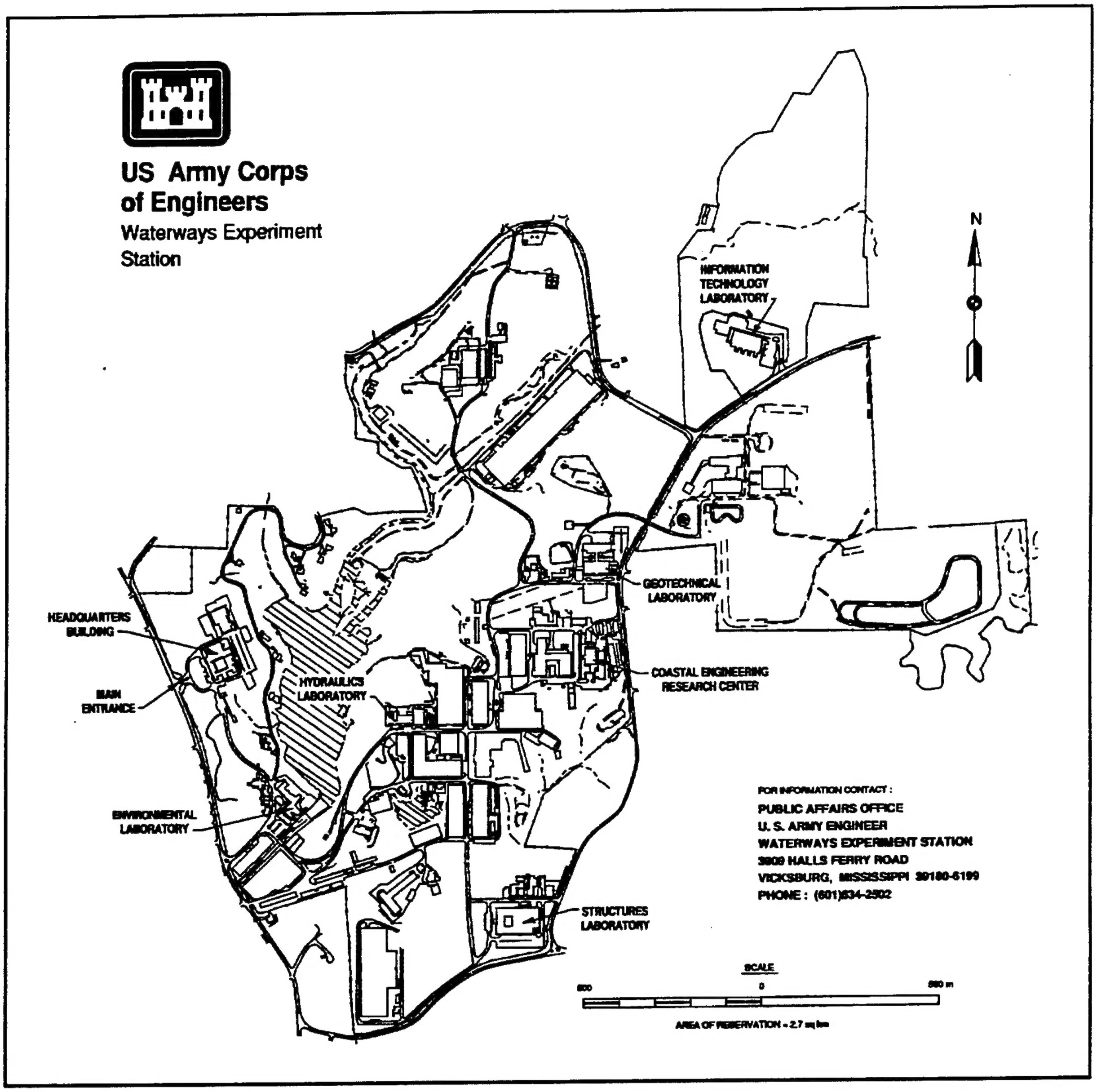
U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Accesion For	
NTIS CRA&I	✓
DTIC TAB	☐
Unannounced	☐
Justification	
By	
Distribution /	
Availability Codes	
Dist	Avail and / or Special
A-1	

Report 1 of a series

Approved for public release; distribution is unlimited

Prepared for U.S. Army Engineer District, New Orleans
P.O. Box 60267, New Orleans, LA 70160-0267



Waterways Experiment Station Cataloging-in-Publication Data

Redeye Crossing Reach, Lower Mississippi River. Report 1, Sediment investigation / by T.J. Pokrefke ... [et al.] ; prepared for U.S. Army Engineer District, New Orleans.
78 p. : ill. ; 28 cm. — (Technical report ; HL-95-13 rept.1)
Includes bibliographical references.
Report 1 of a series.
1. Sediment control — Mississippi River. 2. Dikes (Engineering) — Mississippi River. 3. Mississippi River. I. Pokrefke, Thomas J. II. United States. Army. Corps of Engineers. New Orleans District. III. U.S. Army Engineer Waterways Experiment Station. IV. Hydraulics Laboratory (U.S. Army Engineer Waterways Experiment Station) V. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-95-13 rept.1.
TA7 W34 no.HL-95-13 rept.1

Contents

List of Figures

Preface

The Model investigation reported herein was conducted for the U.S. Army Engineer District, New Orleans (LMN), in the Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, during the period from April 1989 to October 1991. The investigation was conducted under the general supervision of Mr. F. A. Herrmann, Jr., Director, Hydraulics Laboratory (HL), and under the direct supervision of Mr. M. B. Boyd, Chief, Waterways Division, HL. The engineer in immediate charge of the physical model investigation was Mr. T. J. Pokrefke, Jr., Chief, River Engineering Branch, Waterways Division. Mr. Pokrefke was assisted by Messrs. C. R. Nickles and B. T. Crawford, River Engineering Branch. The engineer in immediate charge of the numerical model investigation was Mr. M. J. Trawle, Chief, Math Modeling Branch, Waterways Division. The Project Engineer for the numerical model investigation was Mr. Nolan K. Raphelt with engineer support provided by Mr. Gary E. Freeman and Mrs. Lisa L. Benn. Technical consultation was provided by Mr. W. A. Thomas. Technician support was provided by Ms. Brenda L. Martin. The physical model segment of the report was prepared by Messrs. Nickles and Pokrefke, and the numerical model segment was prepared by Messrs. Raphelt and Trawle. Consolidation of the material into a comprehensive report on the sediment investigation was coordinated by Mr. Boyd.

During the course of the model study, LMN was kept informed of the progress of the study through monthly progress reports and interim tests results. Messrs. C. Soileau, B. Garrett, and Ms. N. Powell, LMN, visited WES to observe model tests, discuss test results, and coordinate the testing program.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

The contents of this report are not to be used for advertising, publication, or promotional purposes. Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
miles (U.S. nautical)	1.852	kilometers
miles (U.S. statute)	1.609347	kilometers

1 Introduction

Description of Problem

Redeye Crossing is located on the lower Mississippi River between river miles 223 and 225, Above Head of Passes (AHP), about 3 miles[1] downstream of the I-10 Highway Bridge at Baton Rouge, LA (Figure 1). The existing condition at Redeye Crossing, without benefit of any training structures or constricting dikes, requires approximately 3,000,000 cu yd of dredging annually to maintain the 40-ft-deep draft navigation channel through the crossing. The crossing shoals during rising river stages and requires the channel to be dredged when the stages start receding to maintain a 40-ft-deep draft navigation channel to Baton Rouge. A proposed 45-ft-depth navigation channel to Baton Rouge would drastically increase maintenance dredging requirements unless corrective measures were implemented.

Typically, the stage at Redeye Crossing varies over a range of about 30 ft during a water year. Average water velocities in the crossing range from about 3 fps during low-water stages to about 6 fps during high-water stages. Without maintenance dredging, the controlling depths at the crossing would be less than 30 ft during low-water periods. The bed sediment at the crossing is primarily sand with a D50 size of about 0.25 mm.

Purpose of Study

The purpose of the Redeye Crossing Reach study was to evaluate the effectiveness of proposed constricting dikes at Redeye Crossing in reducing maintenance dredging requirements while maintaining safe navigation conditions through the reach. This report will discuss the sedimentation investigations conducted to evaluate the effectiveness of the dikes in reducing dredging requirements. Report 2 will discuss the Ship/Tow Simulator investigation of navigation conditions in the Redeye Crossing Reach.

[1] A table of factors for converting non-SI units of measurement to SI units is presented on page vi.

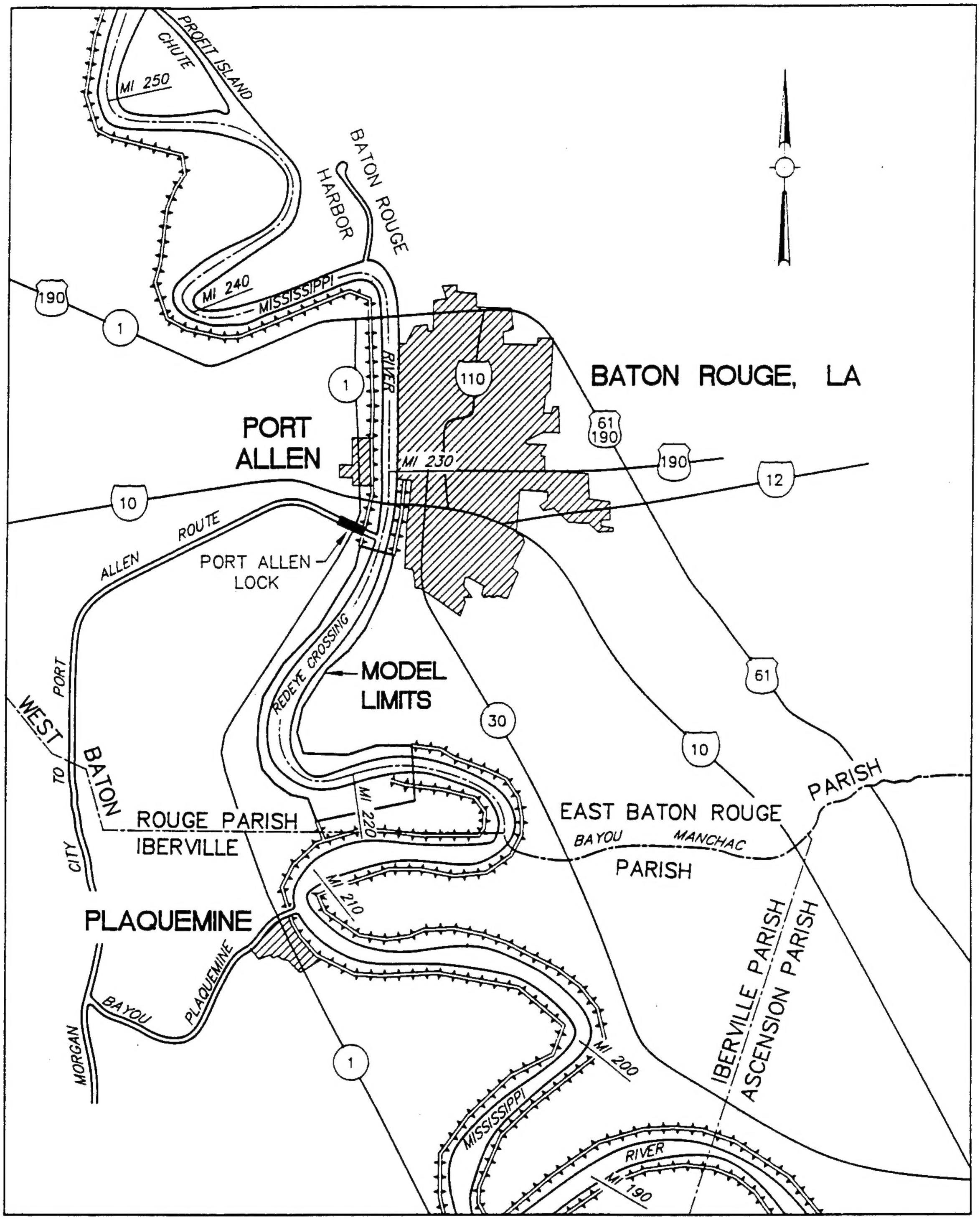


Figure 1. Vicinity map

Study Approach

The sedimentation study was conducted using both physical and numerical modeling techniques, taking advantage of the capabilities of both types of models. While the physical movable-bed model was being constructed and verified, one- and two-dimensional (1-D, 2-D) numerical models were used to determine an approximate contraction width and preliminary dike spacing and location. Since no dikes presently exist in this portion (deep-draft channel) of the Mississippi River, this approach was developed to provide an improved plan for initial tests in the physical model. The 1-D numerical sedimentation model (TABS-1) developed for investigating the effect of flow diversions in the Lower Mississippi River was refined in the Redeye Crossing Reach and used to estimate the required contraction. It also provided boundary conditions for the 2-D numerical modeling system (TABS-2) used for more detailed modeling of a shorter reach through Redeye Crossing. The TABS-1 work at Redeye Crossing was included in the report on the flow diversion study.[1] The TABS-2 modeling system was used in hydrograph simulations of proposed dike plans as a screening process to minimize the required testing program in the physical movable-bed model.

In the winter and spring of 1991, interest in constructing dikes during the 1991 construction season to reduce maintenance dredging for the existing 40-ft channel resulted in additional numerical model tests to provide information contributing to dike design decisions (including predicted channel cross sections and velocity fields for use in navigation simulation studies in the U.S. Army Engineer Waterways Experiment Station (WES) Ship/Tow Simulator). This interest also led to termination of physical model verification refinements and an abbreviated physical model testing program. Ultimately, concern expressed by navigation interests over the use of rock dikes precluded dike construction in 1991. However, this situation did require adjustments in the planned study approach.

Assessment of Available Prototype Data

Prototype data indicate the crossing shoals during rising river stages and require the channel to be dredged when the stages start receding to maintain a 40-ft-deep draft navigation channel to Baton Rouge. These data indicate that the crossing can fill as much as 10 ft during an event. Usually maintenance dredging removes the shoal material during the falling side of the hydrograph, but the channel will normally fill again during the next rising river stages.

In the same general vicinity as Redeye Crossing, the tendencies in the bendway immediately downstream, Missouri Bend, are rather unusual. In an

---

1 Ronald R. Copeland and William A. Thomas. (1992). "Lower Mississippi River Tarbert Landing to east jetty sedimentation study; numerical model investigation," Technical Report HL-92-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS. 39180.

alluvial river it is normally expected that areas such as the elevations on the point bar at Missouri Bend are hydrograph dependent. Therefore, one would expect that after extended high water the point bar would be at a higher elevation than before, and on the falling and intermediate-to-low stages the bar would normally scour. Numerous prototype surveys were reviewed, and one conclusion reached was that the elevation and size of the point bar at Missouri Bend is extremely stable relative to elevation and areal extent. On the point bar for a distance of about 2,500 ft from the left bank from about mile 222.7 to 221.5, but not including the towhead at mile 221.7, the bed elevations changed very little over time. This is true regardless on the timing of the survey. For example, in July 1983 following a high water, the average or general elevation of the point bar in this area was about the same as after an extended low water in November 1987. These tendencies, with significant shoaling in Redeye Crossing during high flows and no major changes in the point bar at Missouri Bend, complicate modeling of the reach.

2 Numerical Model Investigation

Model Information

Description

The 2-D numerical model study was conducted using the TABS-2 modeling system.[1] This system provides 2-D solutions to open-channel and sediment problems using finite element (FE) techniques. The system consists of more than 40 computer programs to perform modeling and related tasks. A 2-D depth-averaged hydrodynamic numerical model, RMA-2V, was used to generate the current patterns. The current patterns were then coupled with the sediment properties of the river and used as input to a 2-D sedimentation model, STUDH. The other programs in the system perform digitizing, grid generation, data management, graphical display, output analysis, and model interfacing tasks. The sediment model requires hydraulic parameters from RMA-2V, sediment characteristics, inflow concentrations, and sediment diffusion coefficients. Sediment is represented by a single grain size, and transport potential is calculated using the Ackers-White equation.[2]

Finite element grids

Finite element grids were developed to simulate the Mississippi River from river mile 228 AHP to river mile 206 AHP, representing a distance of 22 miles. The overall grid was modified to accomodate dike plans only within the Redeye Crossing Reach of the model. The upstream and downstream portions of the grid were identical for all testing. Initial bed elevations were obtained from the 1986 hydrographic surveys. A typical model grid of the

---

1 Thomas, W. A., and McAnally, W. H., Jr. (1985). "User's manual for the generalized computer program system; open-channel flow and sedimentation, TABS-2, main text and appendices A through O," Instruction Report HL-85-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

2 Ackers, P., and White, W. R. (1973). "Sediment transport: New approach and analysis," Journal, Hydraulics Division, American Society of Civil Engineers, 99(HY-11), 2041-2060.

entire 22-mile reach is shown in Figure 2. The area modified to accommodate dike plans is indicated by the boxed portion at Redeye Crossing (Figure 2).

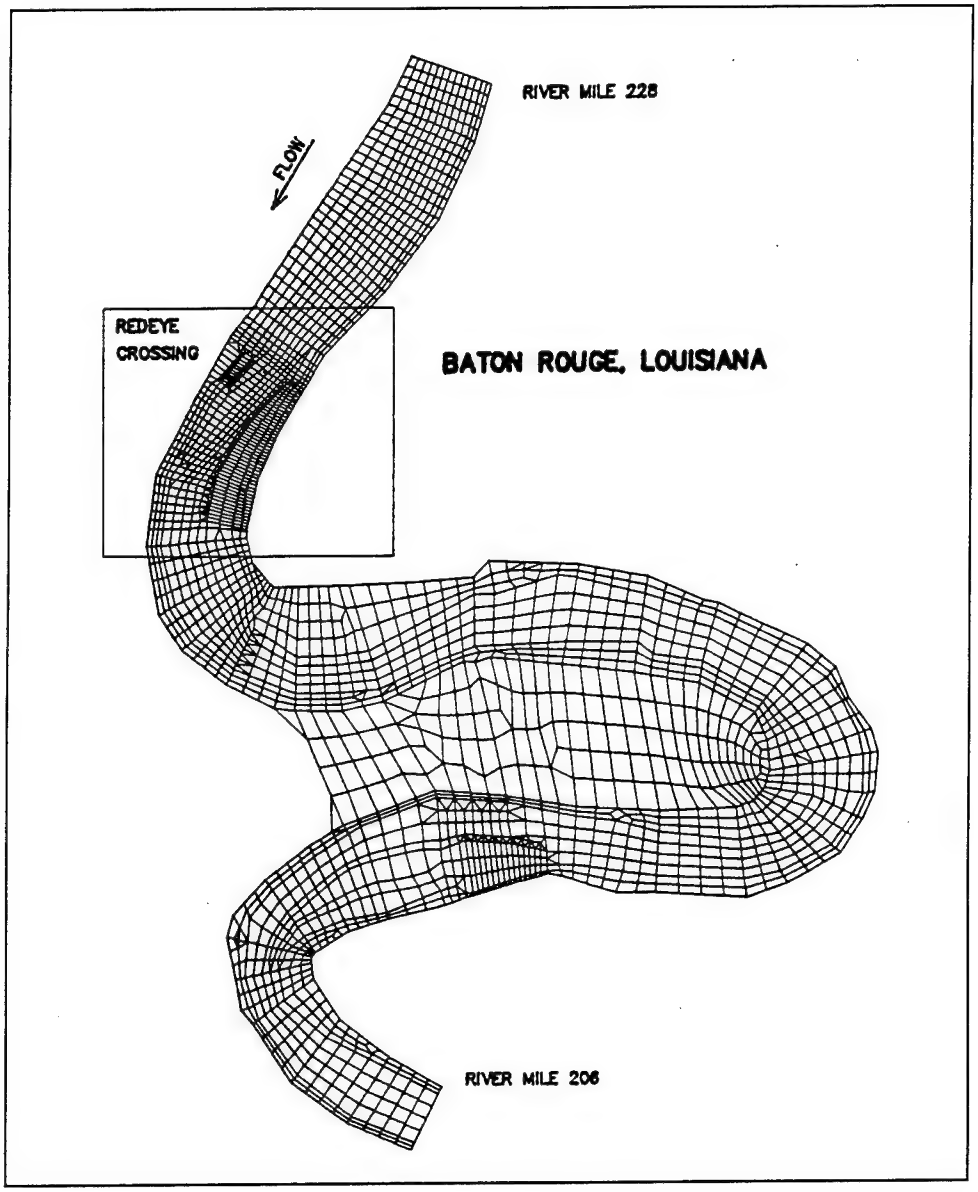


Figure 2. Numerical model (TABS-2) grid

The Redeye Crossing portion of the Existing Condition and Plan 1 grids is shown in Figure 3. The Existing Condition and Plan 1 grids consisted of 2,730 elements and 8,114 nodes. The only difference in these grids was the bed elevations at the dike locations. In the Existing-Condition grid, the nodes associated with the dike elements were assigned Existing-Condition elevations. In the Plan 1 grid, the nodes associated with the dike elements were "popped up" by assigning dike crest elevations.

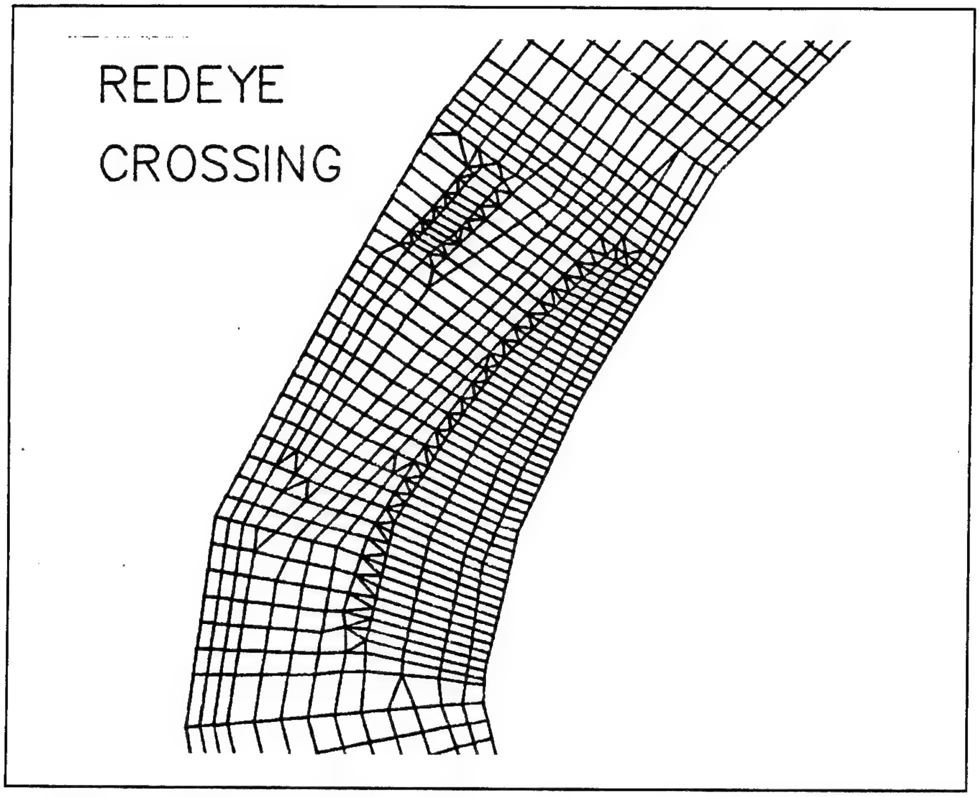


Figure 3. Redeye Crossing portion of TABS-2 grid

Plans 2, 3, 4, 5, and 6 grids consisted of 2,742 elements and 8,172 nodes. The only difference in these grids was dike crest elevations. In plan view, they were all identical.

The Plan 7 grid consisted of 2,745 elements and 8,176 nodes. In plan view, the only difference between Plan 7 and Plan 6 grids was that the dikes on the right descending bank were extended 200 ft riverward.

The Plan 8 grid consisted of 2,748 elements and 8,182 nodes. In plan view, the only difference between Plan 8 and Plan 7 grids was that the lower three dikes on the left descending bank were extended 200 ft riverward.

In plan view, Plans 5A, 5AO, and 5BO grids were identical to Plan 5 and Plans 8A, 8AO, and 8BO grids were identical to Plan 8. Differences were limited to the number of dikes which were "popped up" and their elevations.

Hydrodynamic boundary conditions

Each plan was tested using at least one of three different hydrodynamic boundary conditions. For each condition, discharges were specified at the upstream boundary and water levels (stage) at the downstream boundary. The stages used were interpolated from prototype data at existing river gauges.

The three approaches consisted of steady-state discharge modeling over a range of discharges; step-dynamic discharge modeling using the 43-year-average-annual hydrograph; and dynamic discharge modeling using the 1990 observed hydrograph.

Steady state boundary conditions. The Existing Condition and Plans 1, 6, and 7 were subjected to a steady-state analysis. The discharges used and the period of time simulated are given in the following tabulation.

Discharge, cfs	Downstream elev, ft (NGVD[1]	Time, days
750,000	24.4	30
1,000,000	30.1	30
1,250,000	35.2	30
1,500,000	40.1	30

[1] All elevations (el) and stages cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

Step-dynamic boundary conditions. The Existing Condition and a number of the plans were dynamically tested using the 43-year-average-annual discharge hydrograph at the upstream boundary (Plate 1) and stage downstream (based on the stage-discharge rating curve shown in Figure 4). The length of these simulations was typically one or two cycles of the hydrograph (1 or 2 years).

Dynamic boundary conditions. The Existing Conditions and some of the plans were tested using the observed 1990 hydrograph at the upstream boundary (Figure 5) in a fully dynamic fashion. The downstream boundary (stage) was handled using the stage-discharge rating curve (Figure 4). The length of these simulations was intended to represent about 8 months, but code stability problems limited the simulations to the first 70 days.

Roughness coefficients. Within the study reach, Manning's n values ranged from 0.016 in the main river channel to 0.10 over the submerged dikes.

Turbulent exchange coefficients. Within the study reach, the turbulent exchange coefficient used ranged from 30 to 80 lb-sec/ft.

Sediment transport boundary conditions

The boundary information required by STUDH was suspended sediment concentrations at the upstream boundary and bed sediments within the model. Primary input parameters required by STUDH were dispersion coefficients, effective particle diameter for transport, effective settling velocity, and Manning's n for bed shear stress.

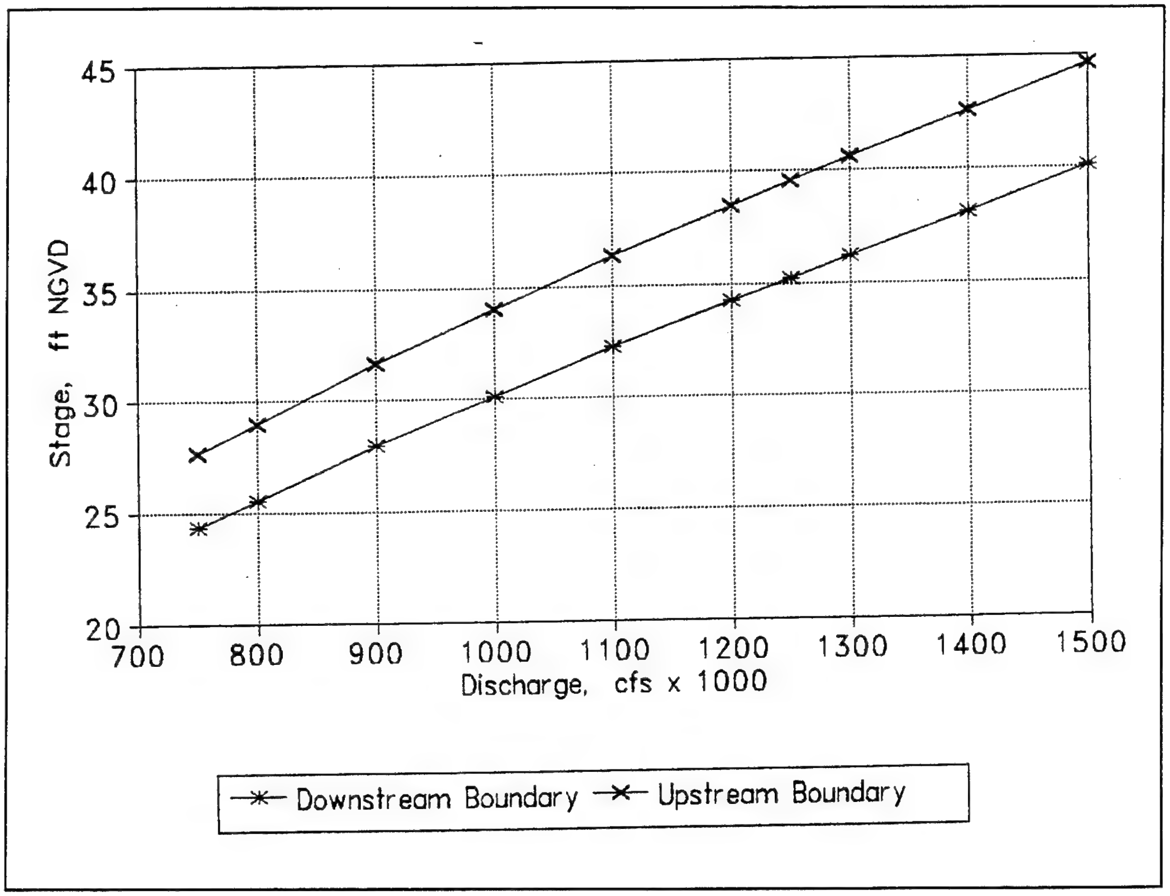


Figure 4. Stage-discharge rating curves at upstream and downstream model limits

Suspended sediment concentration. Suspended sediment concentrations used at the upstream boundary were generated from results developed by the existing 1-D long-term-trend sediment transport model.[1] The sediment discharge rating curve for medium sand, developed from the 1-D work, is shown in Figure 6.

Bed sediments. Based on the gradation analysis of sediment samples taken from the Redeye Crossing Reach, model bed sediment used in STUDH was 0.25 mm, representing a medium sand bed.

Dispersion coefficients. Within the study reach, the dispersion coefficients used in STUDH ranged from 5 to 15 m^2/sec.

Effective settling velocity. The effective settling velocity used in STUDH was 0.03 m/sec for all tests.

---

[1] Ronald R. Copeland and William A. Thomas. (1992). "Lower Mississippi River Tarbert Landing to east jetty sedimentation study; numerical model investigation," Technical Report HL-92-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

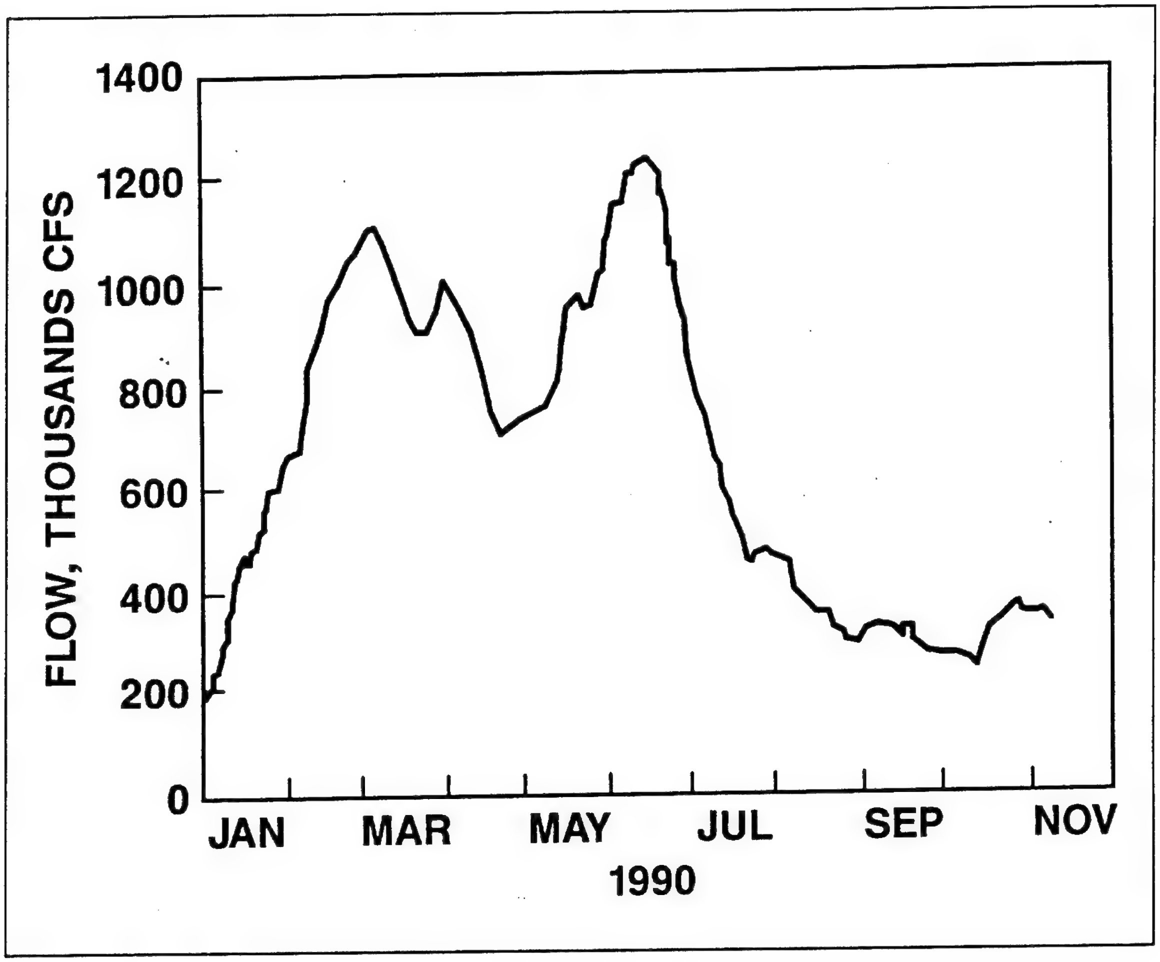


Figure 5. 1990 discharge hydrograph

Particle diameter for transport. The particle diameter for transport was set at 0.25 mm for all tests.

Manning's n for bed shear stress. Manning's n for bed shear was the same as used in the hydrodynamic code.

Model adjustment

Hydrodynamic adjustment. Because of the lack of prototype velocity data, the adjustment procedure was basically limited to water-level slope over the length of the modeled reach of river.

The primary adjustment parameters required by the hydrodynamic code (RMA-2V) as model input were Manning's n values and turbulent exchange coefficients. These parameters were adjusted within reasonable limits until water-surface-slope profiles in the model agreed with observed profiles in this reach of the river.

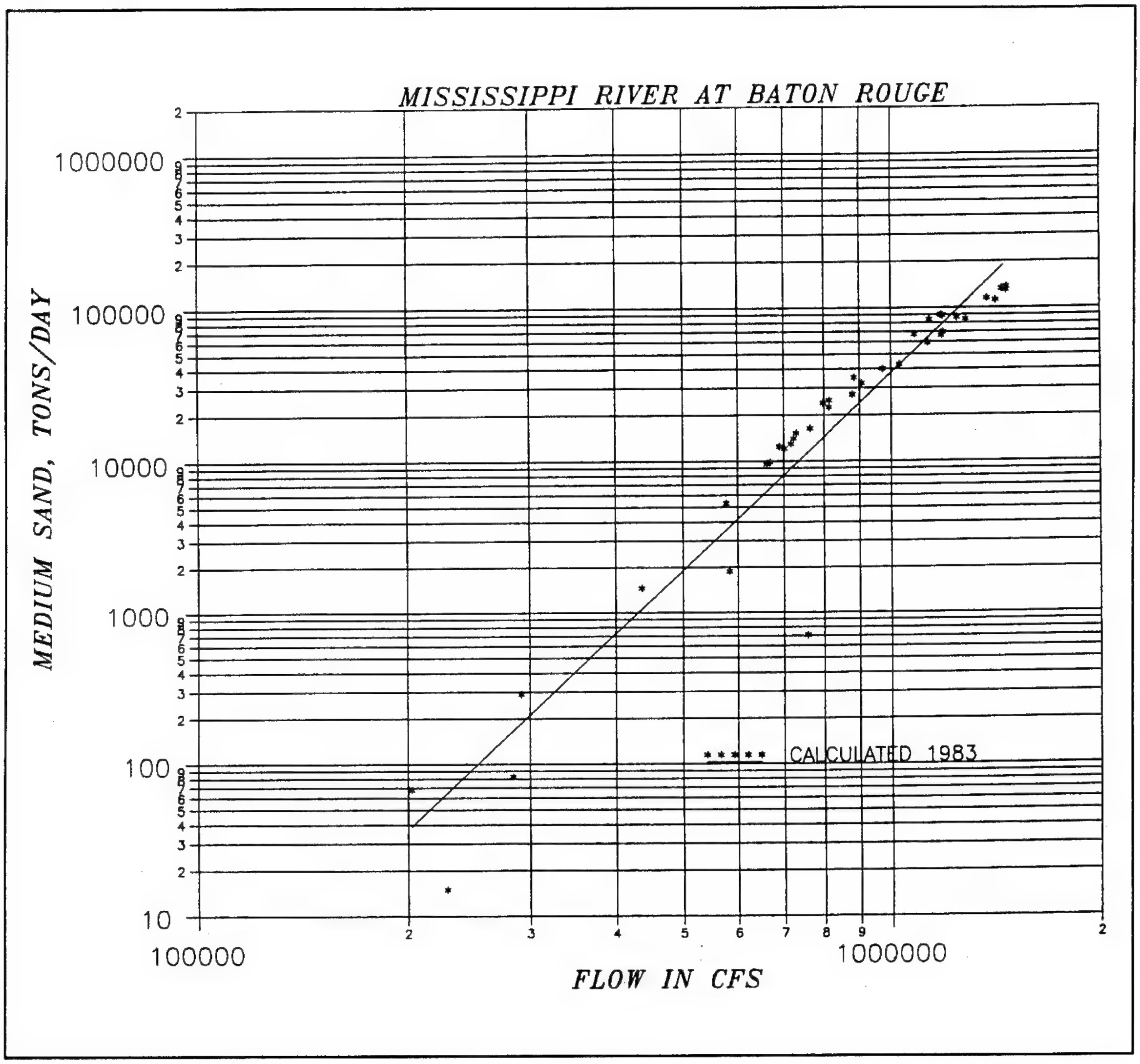
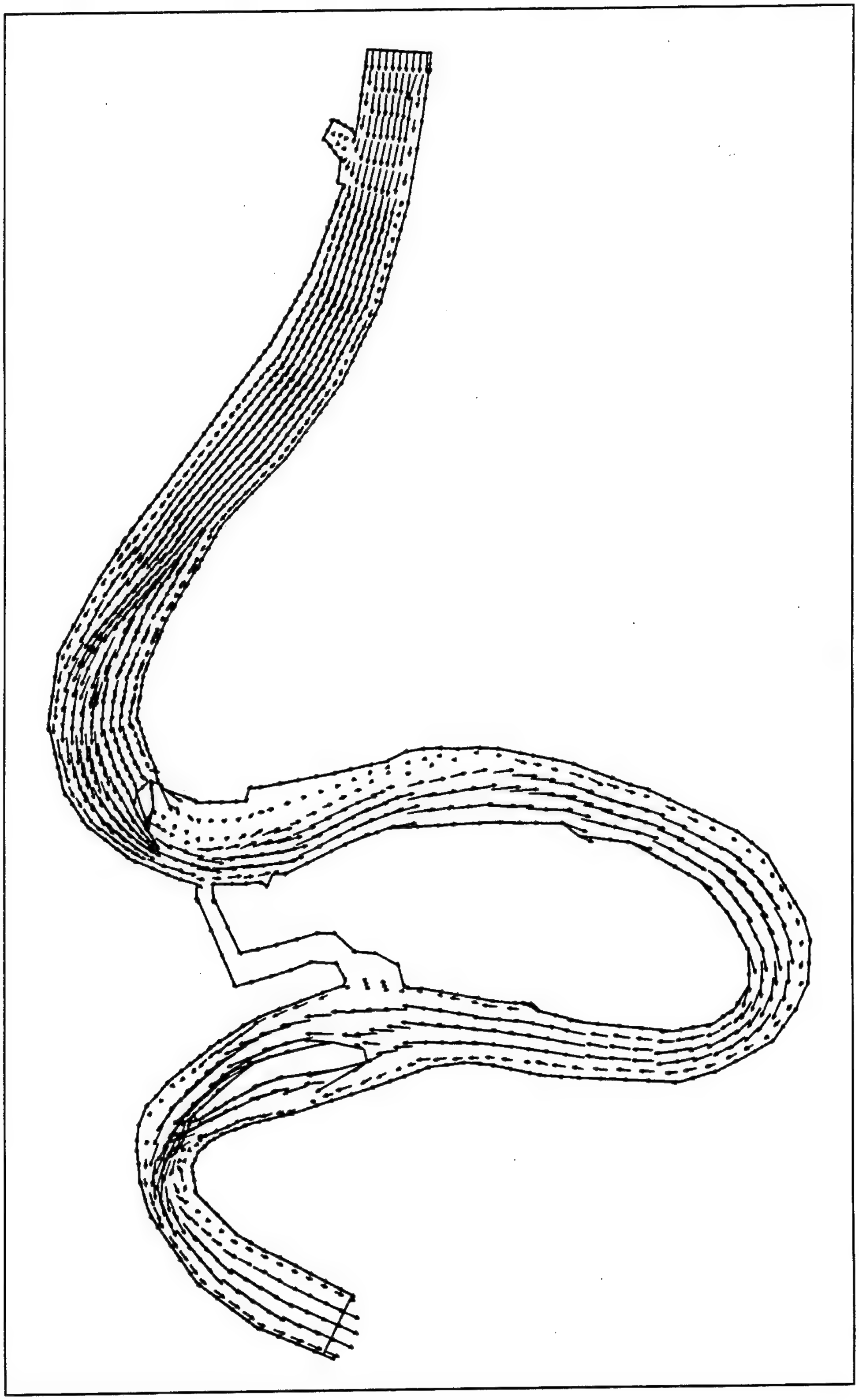


Figure 6. Sediment (medium sand) discharge rating curve

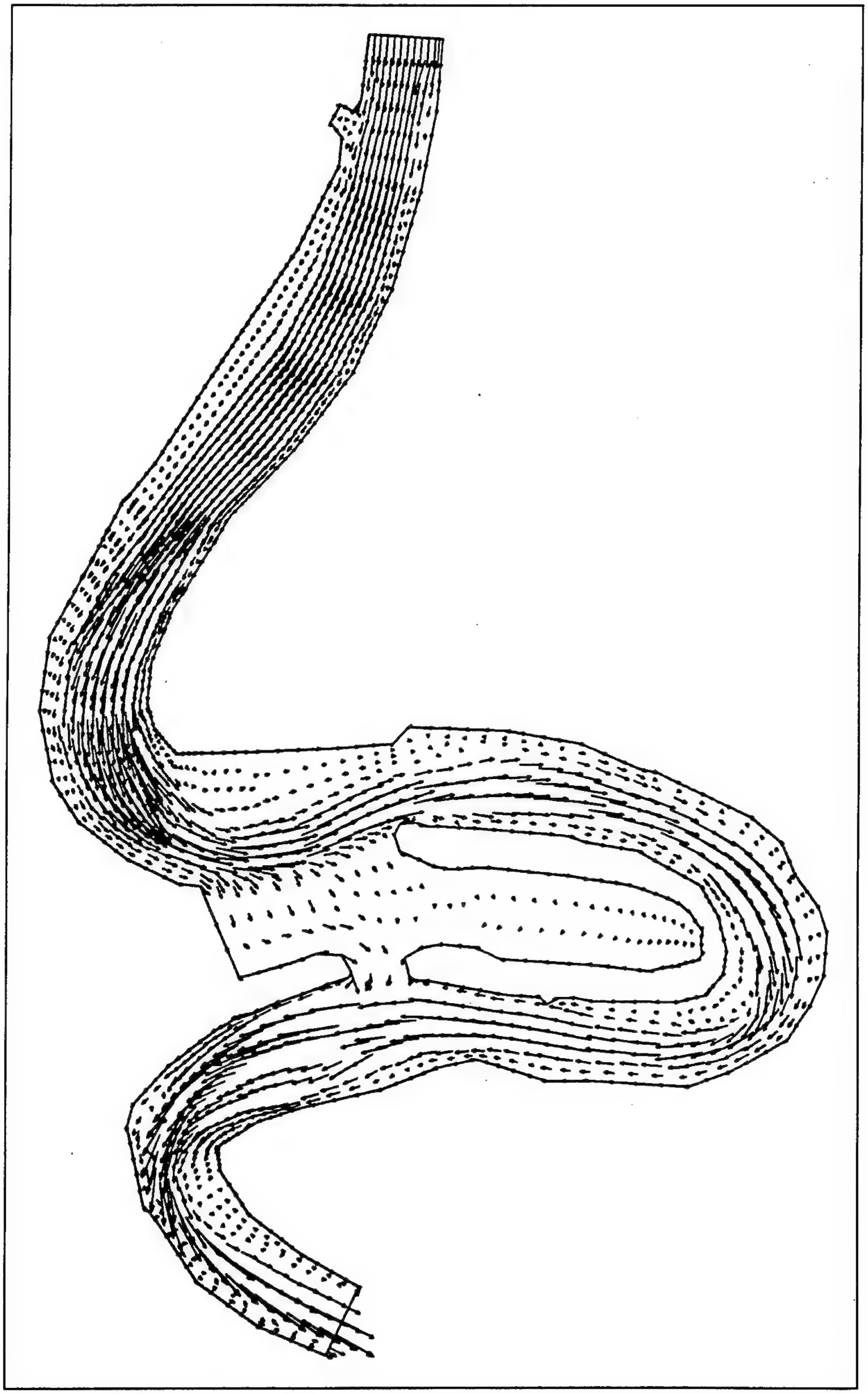
Flow fields generated by the model appeared reasonable. Examples of existing-condition flow fields for the 750,000 and 1,500,000 cfs discharges are shown in Figures 7a and 7b, respectively.

Sedimentation adjustment. The basic procedure in adjusting the sediment transport code, STUDH, involved the comparison of model sedimentation volumes in the Redeye Crossing channel over the 43-year-average-annual hydrograph to reported maintenance dredging volumes. The procedure assumes that dredging volumes can be equated to deposition volumes.



a. Discharge of 750,000 cfs

Figure 7. Existing-condition flow field from TABS-2



b. Discharge of 1,500,000 cfs

The reported maintenance dredging volumes at Redeye Crossing from 1964 through 1988 are given in the following tabulation.

Reported Maintenance Dredging Volumes, Redeye Crossing

Year	Volume, thousand cubic yards
1964	2,333
1965	3,446
1966	1,015
1967	3,166
1968	2,986
1969	3,198
1970	2,691
1971	926[1]
1972	2,553
1973	5,584
1974	7,863
1975	4,881
1976	1,069[1]
1977	1,810[1]
1978	2,645
1979	1,211
1980	4,098
1981	507[1]
1982	2,663
1983	1,559
1984	Unavailable
1985	1,018
1986	1,013[1]
1987	Unavailable
1988	1,648[1]

[1]Years in which peak daily discharge did not exceed 1,000,000 cfs

Based on the information in the previous tabulation, the average dredging requirement from 1964 to 1988 (excluding 1984 and 1987) at Redeye Crossing was 2,600,000 cu yd. However, the average for the years that did not exceed 1,000,000 cfs peak discharge was only 1,180,000 cu yd.

For model adjustment purposes, the lower of the two averages (1,180,000 cu yd) was the more representative because the model used the 43-year-average-annual hydrograph peak discharge of only 755,000 cfs as upstream boundary information. As an example, a comparison of the 1981 hydrograph to the 43-year-average-annual hydrograph is shown in Figure 8.

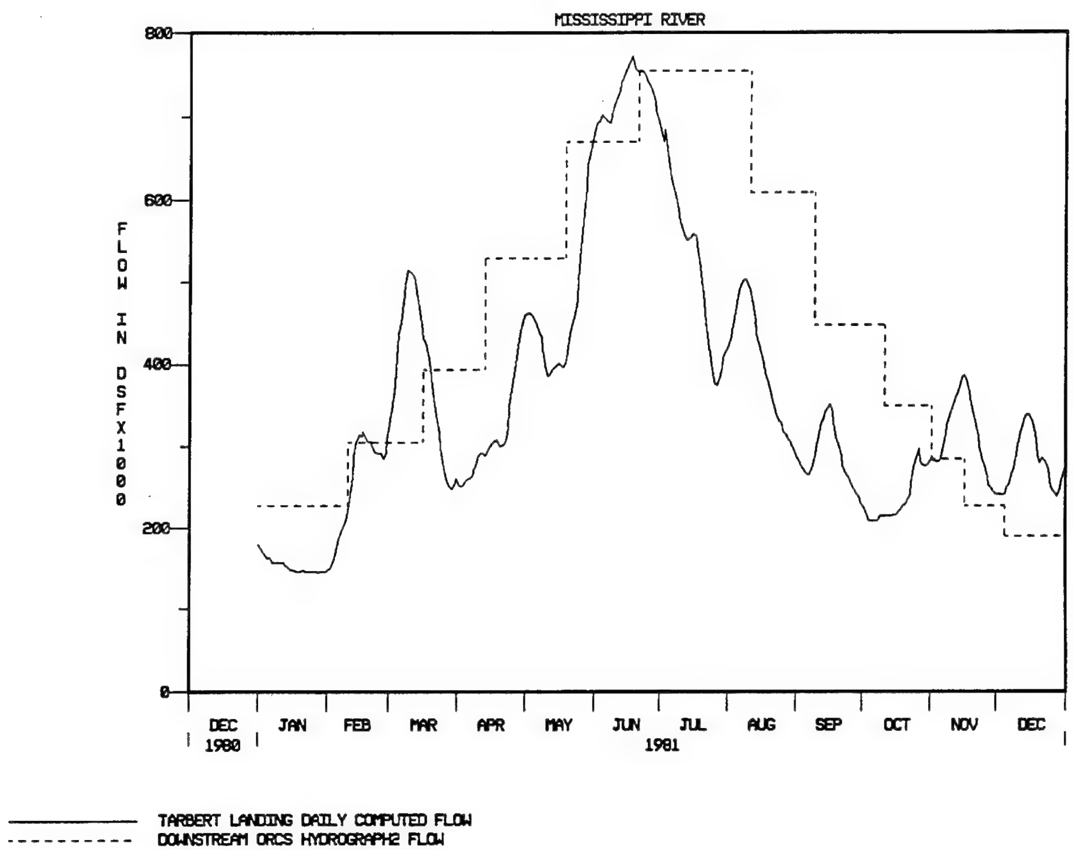


Figure 8. Comparison of 43-year-average-annual and 1981 hydrographs

The primary adjustment parameters required by STUDH were dispersion coefficients, effective particle diameter for transport, effective settling velocity, and Manning's n for bed shear stress.

The model, with existing condition geometry and the 40-ft-deep Redeye Crossing channel installed, was adjusted using the 43-year-average-annual hydrograph, resulting in channel deposition after 1 year of 1,240,000 cu yd (average channel infill of 9.7 ft) which compared well with the average dredging volume of 1,180,000 cu yd discussed above.

The channel area used in the model for calculating volume of deposition is shown in Figure 9. For comparison purposes, the same area was used for all plan testing.

Once the input parameters listed above were set by the existing-condition adjustment testing, they remained fixed for all subsequent plan tests. Additionally, the hydrodynamic code (RMA-2V) computational time-step was fixed at 6 hr, and the sediment transport code (STUDH) time-step was fixed at 15 min for all model base and plan testing.

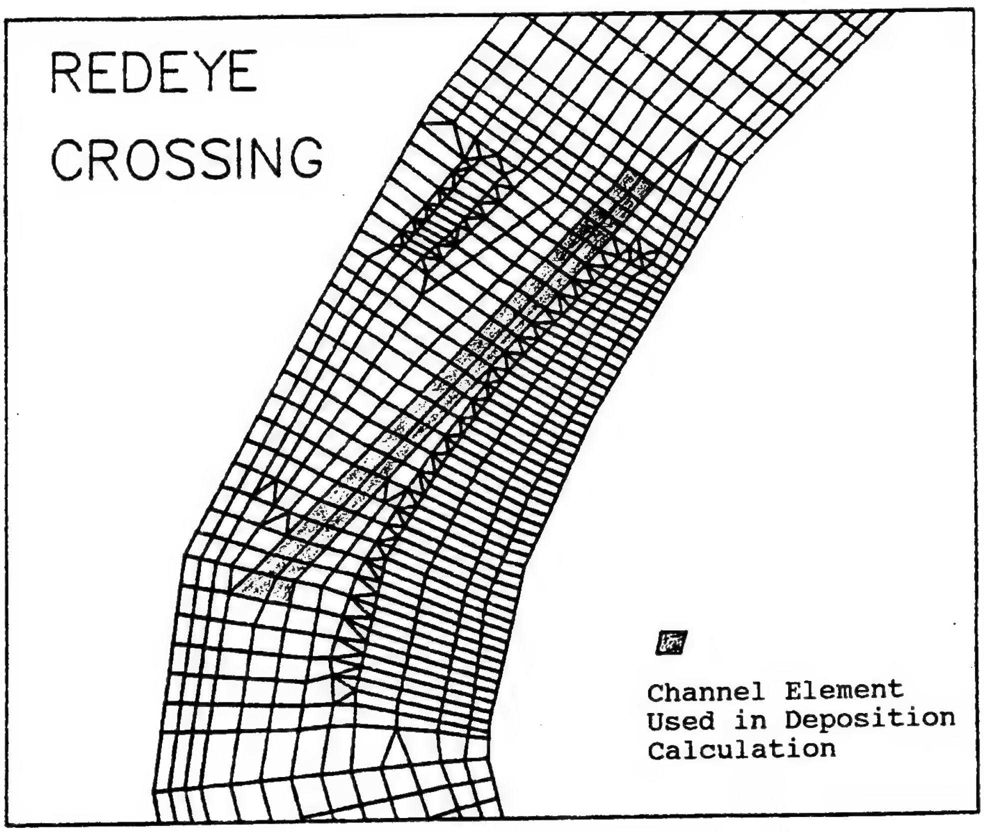


Figure 9. Channel area in TABS-2 used for calculating sediment deposition

Tests and Results

40-ft channel testing

A total of 11 different dike configurations were tested with the 40-ft project, as shown in the following tabulation:

40-ft Project Dike Plan Tests	
Plan	Number of Submerged Lateral Dikes
1	7
2 through 8	9
5A, 5AO	6
5BO	4

The Plan 1 layout is shown in Figure 10. The Plan 2 through 6 layouts, all identical in plan view, are shown in Figure 11. The Plan 7 layout is shown in Figure 12. The Plan 8 layout is shown in Figure 13. The Plan 5A and 5AO layouts, identical in plan view, are shown in Figure 14. The Plan 5BO layout is shown in Figure 15.

Dike crest elevations and lengths. The dike crest elevations for the 40-ft project are as follows:

40-ft Project, Dike Crest Elevations										
Plan	Dike Crest Elevations, ft[1]									
	Left Bank						Right Bank			
Dike No.	1	2	3	4	5	6	1	2	3	4
1	-12	-12	-12				-15	-15	-15	-15
2	-11	-7	-4	-4	0	0	-13	-13	-13	
3	-6	-2	0	0	0	0	-13	-13	-13	
4	-11	-7	-4	-4	0	0	0	0	0	
5	0	0	0	0	0	0	0	0	0	
6	+5	+5	+5	+5	+5	+5	+5	+5	+5	
7	0	0	0	0	0	0	0	0	0	
8	+7	+7	+7	+7	+7	+7	+7	+7	+7	
5A	0	0	0	0	0	0				
5AO	-5	-5	0	0	0	0				
5BO	-5	-5	0	0						

[1] All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD) of 1929.

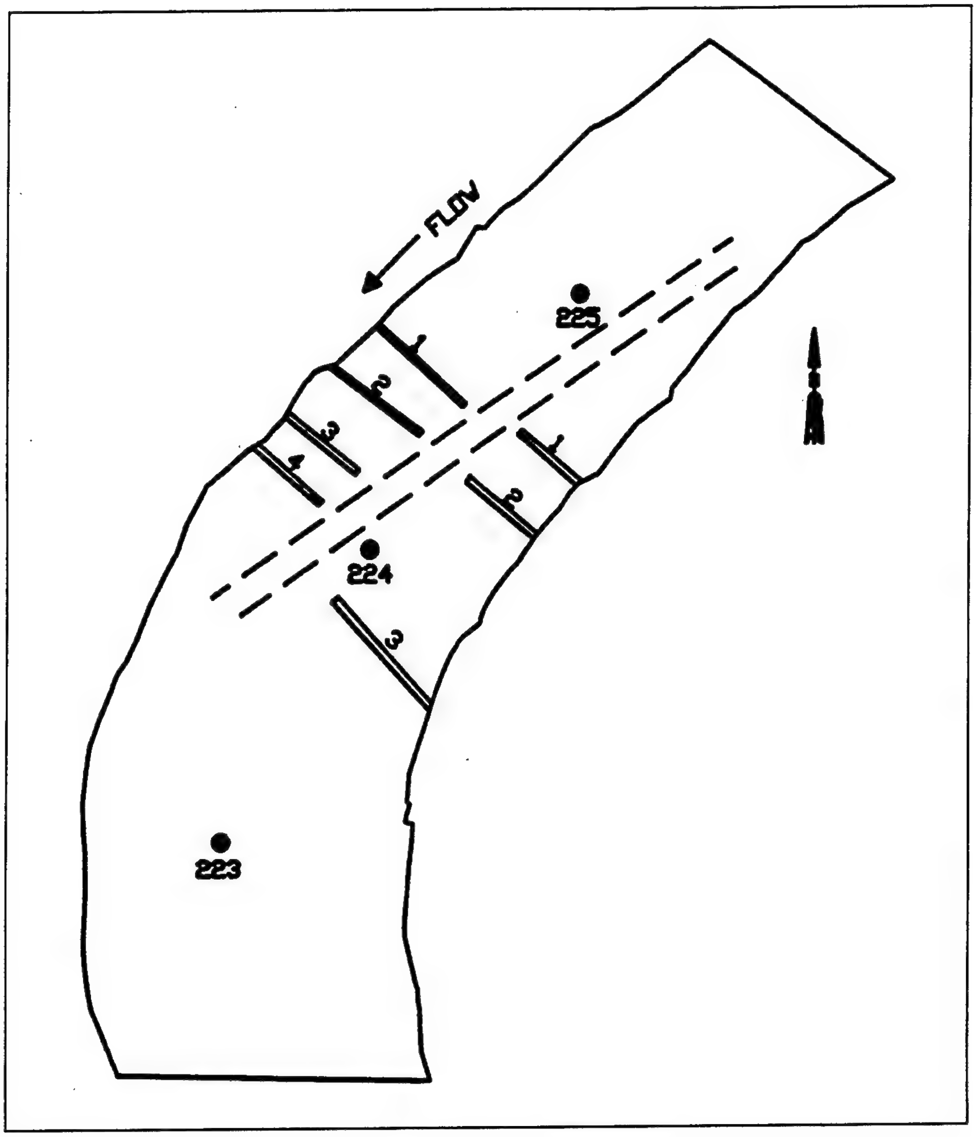


Figure 10. Plan 1 dike layout

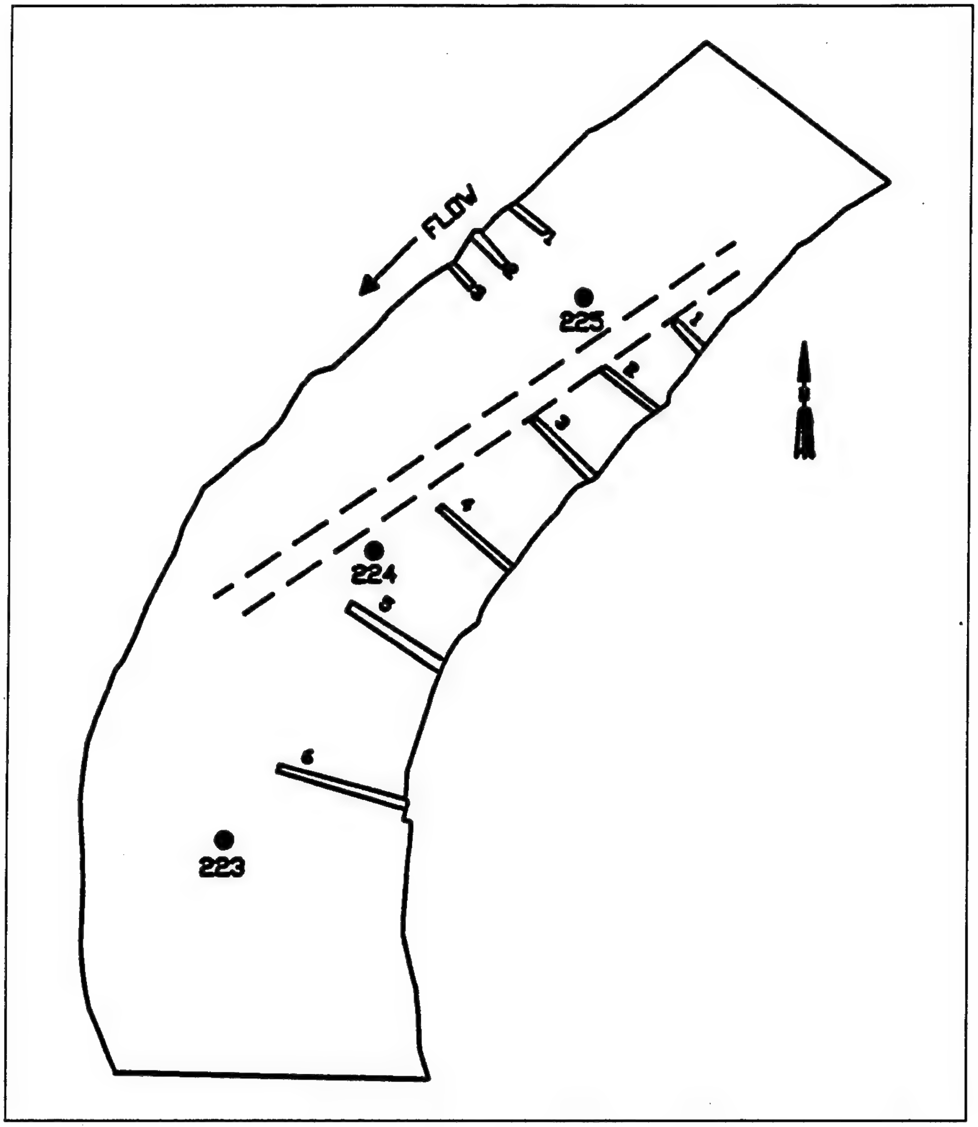


Figure 11. Plans 2, 3, 4, 5, and 6 dike layouts

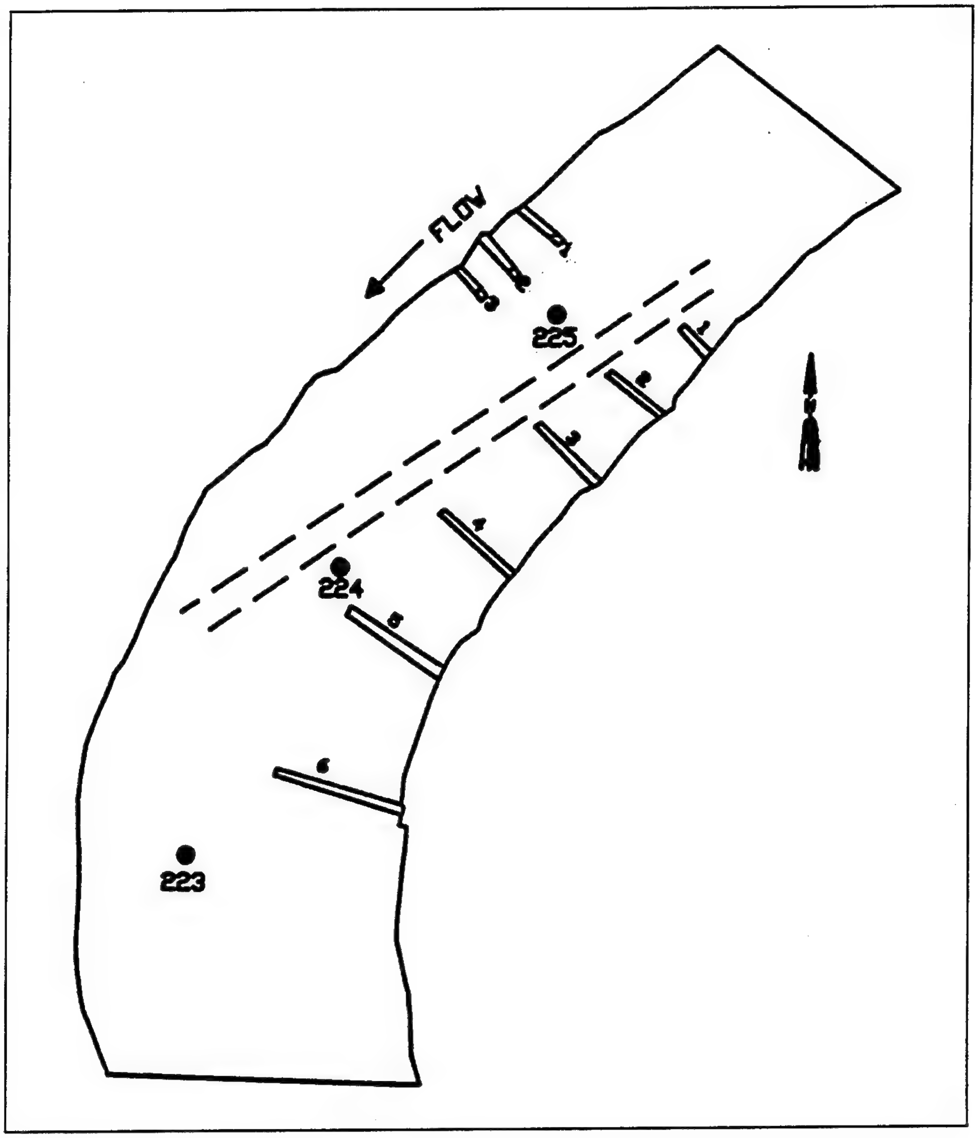


Figure 12. Plan 7 dike layout

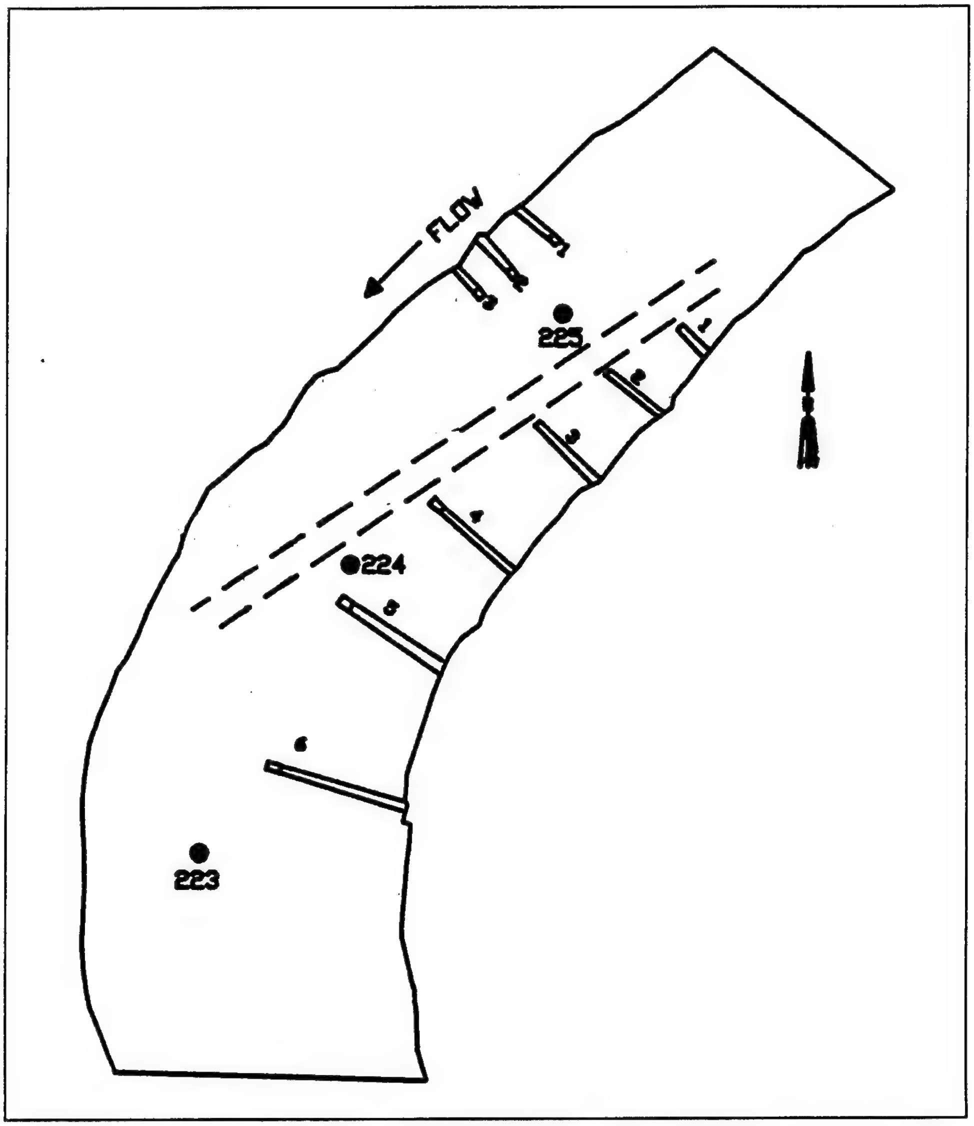


Figure 13. Plan 8 dike layout

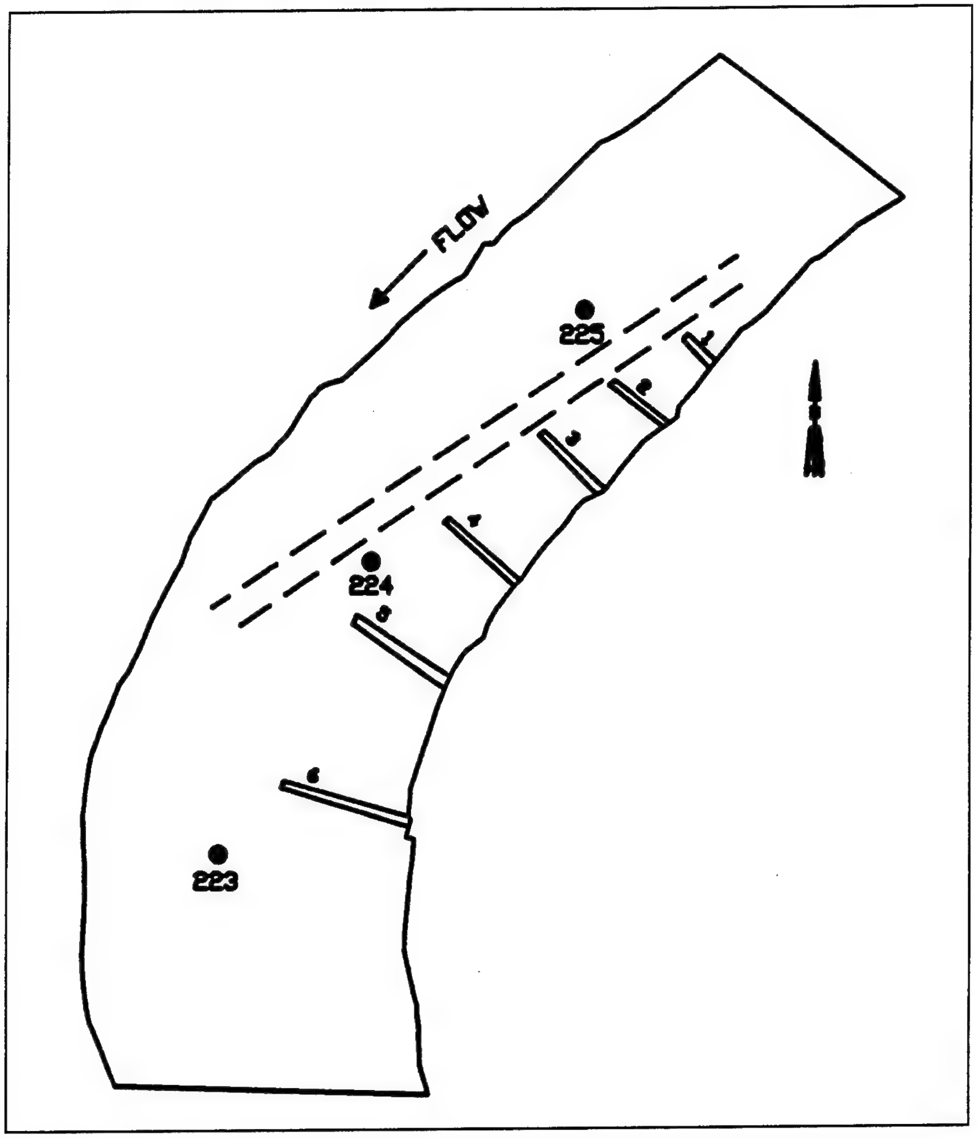


Figure 14. Plans 5A and 5AO dike layouts

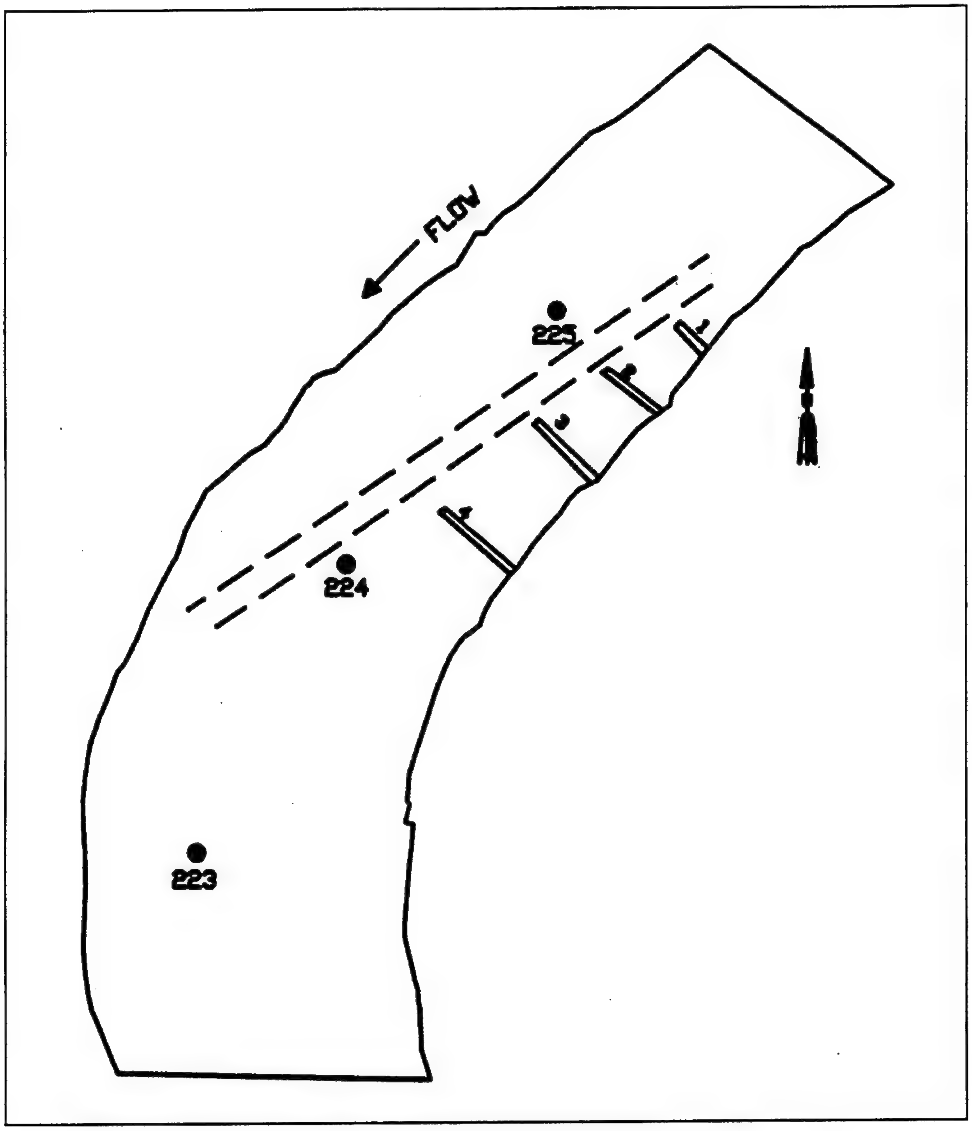


Figure 15. Plan 5BO dike layout

The dike lengths for the 40-ft project are as follows:

	40-ft Project, Dike Lengths									
	Dike Length, ft									
Plan	Left Bank						Right Bank			
Dike No.	1	2	3	4	5	6	1	2	3	4
1	900	1,100	1,700				1,400	1,300	1,100	1,000
2	600	890	1,120	1,070	1,465	1,675	400	400	300	
3	600	890	1,120	1,070	1,465	1,675	400	400	300	
4	600	890	1,120	1,070	1,465	1,675	400	400	300	
5	600	890	1,120	1,070	1,465	1,675	400	400	300	
6	600	890	1,120	1,070	1,465	1,675	400	400	300	
7	600	890	1,120	1,070	1,465	1,675	600	600	500	
8	600	890	1,120	1,270	1,665	1,875	600	600	500	
5A	600	890	1,120	1,070	1,465	1,675				
5AO	600	890	1,120	1,070	1,465	1,675				
5BO	600	890	1,120	1,070						

Hydrographs applied. Each of the above plans was subjected to one or more of the following discharge hydrographs: 43-year-average-annual, 1990, and steady state. The hydrographs applied, and test duration for each of the 40-ft project plan tests are as follows:

Hydrographs Applied and Test Duration		
Plan	Hydrographs Applied	Duration
Base	Steady State,	30 days/discharge,
	43-year-avg-ann, 1990	2 years, 70 days
1	Steady state	30 days/discharge
2	43-year-avg-ann, 1990	1 year, 70 days
3	43-year-avg-ann, 1990	1 year, 70 days
4	43-year-avg-ann, 1990	1 year, 70 days
5	43-year-avg-ann, 1990	2 years, 70 days
6	Steady state	30 days/discharge
7	Steady state	30 days/discharge
8	43-year-avg-ann	1 year
5A	43-year-avg-ann	1 year

Hydrographs Applied and Test Duration (Continued)		
Plan	Hydrographs Applied	Duration
5AO	43-year-avg-ann	150 days
	(high flows only)	
5BO	43-year-avg-ann	150 days
	(high flows only)	

Base and Plan 1 results. Both the Existing Condition (Base) and Plan 1 were subjected to an analysis consisting of steady-state tests using river discharges of 750,000, 1,000,000, 1,250,000, and 1,500,000 cfs. Each steady-state discharge was run for a duration of 30 days and channel deposition determined from the sedimentation code (STUDH). Each discharge test started with the 40-ft project in place. The results are shown in the following tabulation.

Deposition, thousand cubic yards, over 30-day period								
Time in days	Base				Plan 1			
	Discharge, 1,000 cfs				Discharge, 1,000 cfs			
	750	1,000	1,250	1,500	750	1,000	1,250	1,500
10	52	108	167	215	25	64	128	184
20	109	207	317	404	47	136	243	345
30	158	298	455	580	67	197	351	500

The deposition rate (thousand cubic yards per day) calculated from the deposition from day 20 to 30 in above model results are:

Deposition Rate, thousand cubic yards per day		
Discharge	Base	Plan 1
750,000 cfs	4.9	2.0
1,000,000 cfs	9.1	6.1
1,250,000 cfs	13.9	10.8
1,500,000 cfs	17.6	15.4

Psuedo-dynamic results can be obtained by integrating the above deposition rates with selected flow duration data and then summing the results.

Based on observed prototype flow duration data and selected flow intervals, durations for each flow interval can be assigned to the steady-state discharges as follows:

Flow Interval, thousand cfs	Average Discharge, thousand cfs	Days within Interval
625 to 875	750	72
875 to 1,125	1,000	33
1,125 to 1,375	1,250	14
1,375 to 1,625	1,500	4

By integrating flow durations with the calculated volume deposition results, the following psuedo-dynamic comparison between Base and Plan 1 can be made.

Channel Deposition thousand cubic yards		
Discharge, thousand cfs	Base	Plan 1
750	353	144
1,000	300	201
1,250	195	151
1,500	70	62
Total	918	558

These results indicate a Plan 1 reduction in volume deposition at Redeye Crossing of 39 percent.

The Existing Condition (Base) was also tested using the 43-year-average-annual hydrograph and the 1990 hydrograph for comparison to subsequent plan testing. Bed elevations after one and two repetitions of the 43-year-average-annual hydrograph are shown in Plates 2 and 3, respectively. For these tests, the 40-ft project channel was initially excavated at the beginning of the first year and then again at the beginning of the second year. The volume of material excavated after 1 year was 1,240,000 cu yd.

Plan 2 results. Since Plan 1 did not indicate a satisfactory reduction, the dike field was modified by District and WES personnel, resulting in the Plan 2 configuration (Figure 11) with the dike crest elevations as given in this chapter, entitled "Dike crest elevations and lengths." Both the Existing Condition (Base) and Plan 2 were tested in a step-dynamic fashion using the 43-year-average-annual hydrograph (Plate 1) and in a fully dynamic mode using the 1990 hydrograph (Figure 5). The duration for the 43-year-average-annual hydrograph was 1 year, and for the 1990 hydrograph, the duration was only 70 days as indicated in Figure 5. Attempts to run beyond the first 70 days were unsuccessful because of numerical instability problems of the hydrodynamic code RMA-2 beyond 70 days.

The step-dynamic procedure is complex because of the necessity to manually update the model bed elevations at the end of each discharge-step

before running the next step. For the 43-year-average-annual hydrograph, each simulation with a duration of 1 year is actually a series of 12 separate model runs. The updated bed elevations at the end of each discharge-step are passed on to the next step before proceeding with the calculations at the next discharge-step. A flow chart of the operating procedure is given in Figure 16.

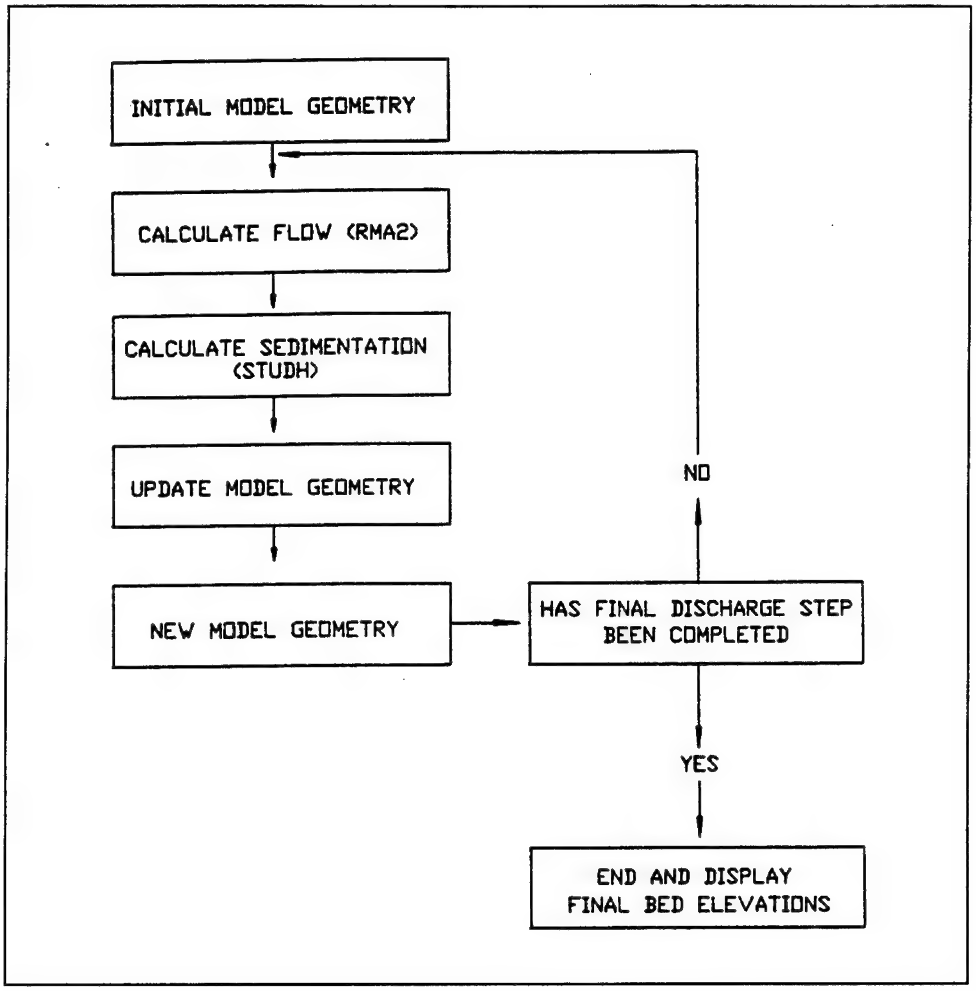


Figure 16. Flow chart of TABS-2 bed update operating procedure

The fully dynamic procedure is even more complex than the step-dynamic mode because discharge values at the upstream boundary change with each computational time-step, with bed updates required at 10-day to 30-day intervals. The bed update interval is a function of river discharge; the greater the discharge, the more frequent the updates. As in the step-dynamic mode, bed updates are done manually, a tedious and time-consuming process. Furthermore, all attempts to run dynamically beyond the first 70 days of the 1990 hydrograph were unsuccessful for the plans tested. The model was not as robust in the fully dynamic mode and required more "tinkering" to maintain numerical stability. Despite an intensive effort to correct the problem, the model became numerically unstable after about 70 days.

Model results in channel deposition for the hydrographs tested are shown:

Channel Deposition, thousand cubic yards			
Hydrograph	Base	Plan 2	Percent Reduction
43-year	1,235	395	68
1990 (70 days)	1,464	585	60

Plan 3 results. Since the model results from Plan 2 did not indicate satisfactory reduction in maintenance dredging requirements, the Plan 2 dike crest elevations were raised (this chapter, paragraph entitled "Dike crest elevations and lengths") to create Plan 3 (Figure 11).

The same testing procedure that was used for Plan 2 was conducted for Plan 3. Model results in channel deposition for the hydrographs tested are as follows:

Channel Deposition, thousand cubic yards			
	Base	Plan 3	Percent Reduction
43-year	1,235	153	87
1990 (70 days)	1,464	624	57

Plan 4 results. Plan 4 (Figure 11) was identical to Plan 3 except that dike crests 1 to 4 were lowered and dikes 7, 8, and 9 were raised as described earlier in this chapter. The dike crest elevations were adjusted in an attempt to improve sediment transport performance beyond Plan 3 performance.

The same testing procedure that was used for Plan 3 was conducted for Plan 4. Model results in channel deposition are shown as follows:

Channel Deposition, thousand cubic yards			
Hydrograph	Base	Plan 4	Percent Reduction
43-year	1,235	178	86
1990 (70 days)	1,464	573	61

Plan 5 results. Plan 5 (Figure 11) was identical to Plan 4 except that dike crests 1 to 4 were raised as described earlier in this chapter. The dike crest elevations were increased to further improve sediment transport performance beyond Plan 4 performance.

The same testing procedure that was used for Plan 4 was conducted for Plan 5.

Model results in channel deposition are shown as follows:

Channel Deposition, thousand cubic yards			
Hydrograph	Base	Plan 5	Percent Reduction
43-year	1,235	102	92
1990 (70 days)	1,464	547	63

Plan 6 and 7 results. Plan 6 (Figure 11) was identical to Plan 5 except that all dike crests were raised an additional 5 ft, as described earlier in this chapter. Plan 6 testing consisted only of steady-state runs designed to check sensitivity of channel deposition to further dike crest raising beyond Plan 5 elevations. Raising the dike crests an additional 5 ft beyond Plan 5 did result in slightly improved performance for the 40-ft project.

Plan 7 (Figure 12) was identical to Plan 5 except that the three dikes on the right-descending bank were extended riverward a distance of 200 ft. As with Plan 6, testing consisted only of steady-state runs designed to check sensitivity of channel deposition to dike extensions. Extending the three dikes on the right descending bank riverward 200 ft did not improve performance.

The steady-state results from Plans 6 and 7 were intended to check the sensitivity of Plan 5 deposition to dike heights and dike lengths. Based on these results, Plan 8 was developed and no further tests of Plans 6 and 7 were conducted.

Plan 8 results. Plan 8 (Figure 13) was identical to Plan 7 except that the three lower dikes on the left-descending bank were extended riverward a distance of 200 ft and, as described earlier in this chapter, the dikes were raised 7 ft (to +7 ft).

The 43-year-average-annual discharge hydrograph was applied to Plan 8, with the channel deposition (thousand cubic yards) as shown below.

Channel Deposition, thousand cubic yards			
Hydrograph	Base	Plan 8	Percent Reduction
43-year	1,235	64	95

Plan 8 final bed elevations (with 40-ft project) after 1 year are shown in Plate 4.

Plan 5A results. Plan 5A (Figure 14) was identical to Plan 5 except that the three dikes on the right-descending bank were removed, leaving a total of six dikes. As with Plan 5, the six remaining dikes on the left-descending bank had crest elevations of zero NGVD as described earlier in this chapter.

The 43-year-average-annual discharge hydrograph was applied, with the average channel infill as shown below.

Channel Deposition, thousand cubic yards			
Hydrograph	Base	Plan 5A	Percent Reductiion
43-year	1,235	102	92

Plan 5A final bed elevations after 1 year are shown in Plate 5.

Plan 5AO results. Plan 5AO (Figure 14) was identical to Plan 5A, except that the crests of the upper two dikes on the left-descending bank were lowered 5 feet (to -5 ft NGVD) as described earlier in this chapter.

Because of study deadlines and time constraints, only the higher flow steps of the 43-year-average-annual hydrograph, consisting of about 150 days, were tested for Plan 5AO. This abbreviated testing procedure invalidated any direct comparison to Plan 5A. However, the results suggested that the lowering of the first two upstream dikes by 5 ft had little or no adverse impact on deposition patterns in the upper end of the crossing channel.

Plan 5BO results. Plan 5BO (Figure 15) was identical to Plan 5AO except that the lower two dikes on the left-descending bank were removed, leaving a total of four dikes.

Again, because of the study deadlines and time constraints, only the higher flow steps of the 43-year-average-annual hydrograph, consisting of about 150 days, were tested. The abbreviated procedure invalidated any direct comparison to Plan 5A. However, the results indicated that the removal of the two downstream dikes caused significant deposition to occur in the lower end of the crossing channel.

45-ft navigation channel testing

The primary testing procedure for the 45-ft project channel base and plan testing was to conduct 2-year-long simulations, using two repetitions of the 43-year-average-annual hydrograph. Initially the channel was set to the 40-ft project. At the end of the first year, the channel was reexcavated to the 45-ft project. At the end of the second year the volume of channel deposition for the 45-ft channel base test was 1,620,000 cu yd.

A total of four different dike configurations were tested with the 45-ft project, as shown in the following tabulation:

45-Ft Project Dike Plan Tests	
Plan	Number of Submerged Lateral Dikes
8	9
8A, 8AO	6
8BO	4

The Plan 8 layout is shown in Figure 13. The Plan 8A and 8AO layouts, identical in plan view, are shown in Figure 17. The Plan 8BO layout is shown in Figure 18.

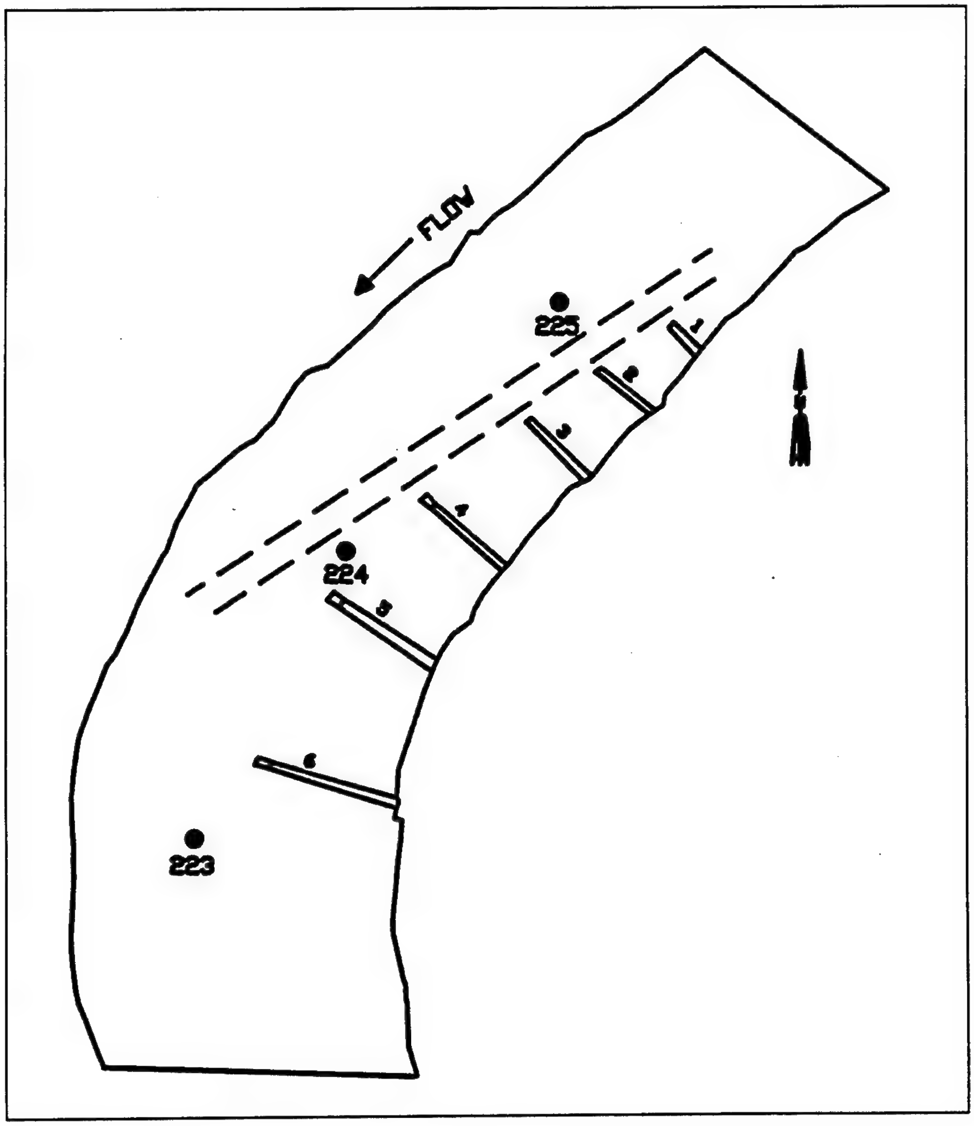


Figure 17. Plans 8A and 8AO dike layouts

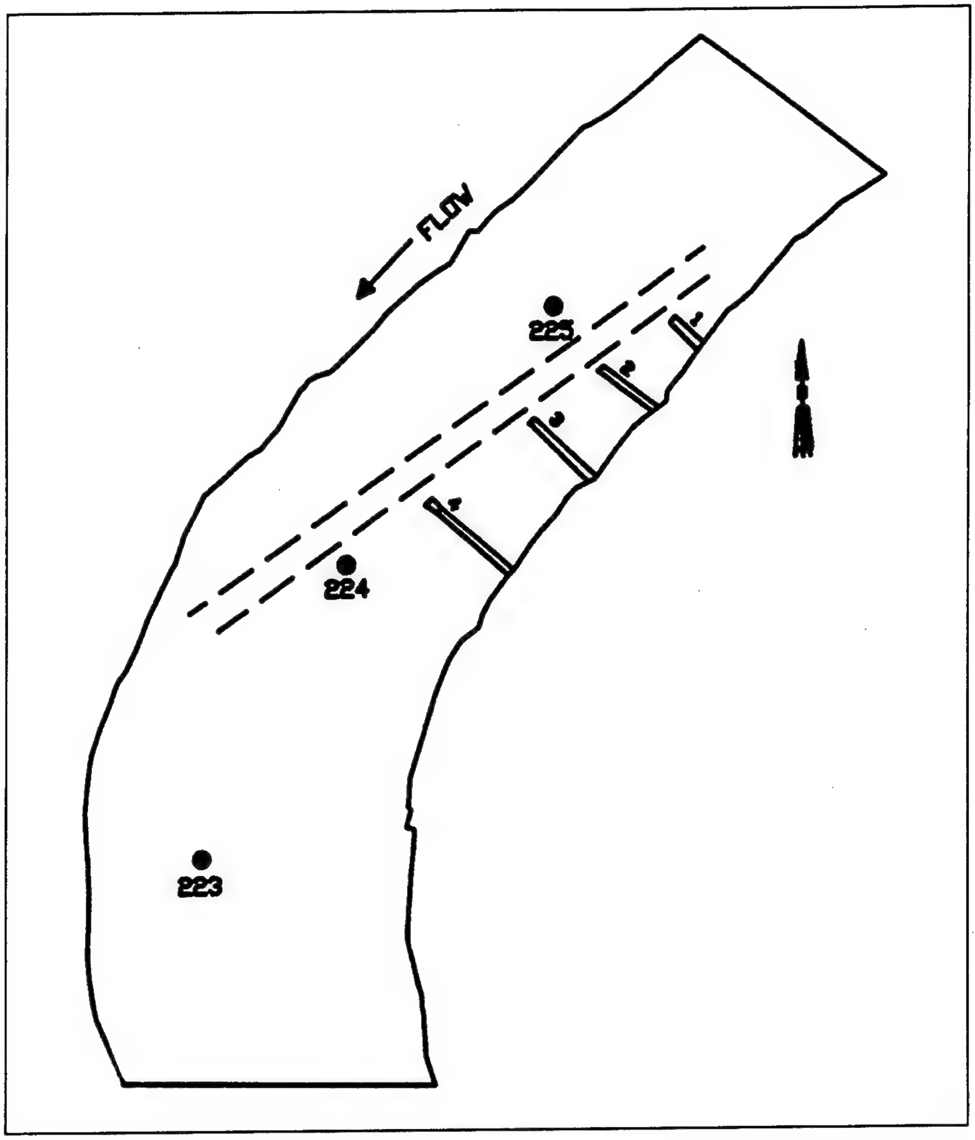


Figure 18. Plans 8BO dike layout

Dike crest elevations and lengths. The dike crest elevations for the 45-ft channel tests are as follows:

45-Ft Project, Dike Crest Elevations									
Plan	Dike Crest Elevation, ft								
	Left Bank						Right Bank		
Dike No.	1	2	3	4	5	6	1	2	3
8	+7	+7	+7	+7	+7	+7	+7	+7	+7
8A	+7	+7	+7	+7	+7	+7			
8AO	+2	+2	+7	+7	+7	+7			
8BO	+2	+2	+7	+7					

The dike lengths for the 45-ft project are as follows:

45-ft Project, Dike Lengths									
Plan	Dike Length, ft								
	Left Bank						Right Bank		
Dike No.	1	2	3	4	5	6	1	2	3
8	600	890	1,120	1,270	1,665	1,875	600	600	500
8A	600	890	1,120	1,270	1,665	1,875			
8AO	600	890	1,120	1,270	1,665	1,875			
8BO	600	890	1,120	1,270					

Plan 8 results. For the Plan 8 testing, the channel was initially excavated to the 40-ft depth. At the end of the first year of the 43-year-average-annual hydrograph, the channel was reexcavated to the 45-ft depth. At the end of the second year the channel deposition was as shown:

Channel Deposition, thousand cubic yards			
Hydrograph	Base	Plan 8	Percent reduction
43-year	1,617	140	91

Final bed elevations after 2 years are shown in Plate 6.

Plan 8A results. Plan 8A dike layout (Figure 17) was identical to the Plan 8 layout except that the three dikes on the right-descending bank were removed, leaving a total of six dikes. As with Plan 8, the six remaining dikes on the left-descending bank had crest elevations of +7 ft (NGVD) as described in this chapter in the paragraph entitled "Dike crest elevations and lengths."

The testing procedure was the same as the Plan 8 procedure. The 43-year-average-annual discharge hydrograph was applied with the channel deposition at the end of the second year as shown:

Channel Deposition, thousand cubic yards			
Hydrograph	Base	Plan 8A	Percent Reduction
43-year	1,617	153	91

Plan 8AO results. Plan 8AO (Figure 17) was identical to Plan 8A except that the crests of the upper two dikes on the left-descending bank were lowered 5 ft (to +2 ft NGVD) as described in this chapter, paragraph entitled "Dike crest elevations and lengths."

Because of study deadlines and time constraints, only the higher discharge steps of the 43-year-average-annual hydrograph, consisting of about 150 days, were tested. This abbreviated testing procedure invalidated any direct comparison to Plan 8A. However, the results suggested that lowering of the first two upstream dikes by 5 ft had little or no adverse impact on deposition patterns in the upper end of the crossing channel.

Plan 8BO results. Plan 8BO (Figure 18) was identical to Plan 8AO except that the lower two dikes on the left-descending bank were removed, leaving a total of four dikes.

Again, because of study deadlines and time constraints, only the higher discharge steps of the 43-year-average-annual hydrograph, consisting of about 150 days, were tested. This abbreviated procedure invalidated any direct comparison to Plan 8A results. However, the results suggested that removal of the two downstream dikes caused significant deposition to occur in the lower end of the crossing channel.

Discussion of Results

The following tabulation summarizes results from the numerical model tests.

	Number of Dikes						
Test Condition	Left Bank	Right Bank	Dike Layout	Dike Elev	Hydrograph	Channel Deposition, thou cu yds	Percent Reduction from Base
40-ft Project Dike Plans							
Base	—	—	—	—	43-year avg	1,235	—
					1990 (70 days)	1,464	—
Plan 1	3	4	Fig 10		Note 1		
Plan 2	6	3	Fig 11		43-year avg	395	68
					1990 (70 days)	585	60
Plan 3	6	3	Fig 11		43-year avg	153	87
					1990 (70 days)	624	57
Plan 4	6	3	Fig 11		43-year avg	178	86
					1990 (70 days)	700	61
Plan 5	6	3	Fig 11		43-year avg	102	92
					1990 (70 days)	547	63
(Continued)							

Test Condition	Number of Dikes: Left Bank	Right Bank	Dike Layout	Dike Elev	Hydrograph	Channel Deposition, thou cu yds	Percent Reduction from Base
40-ft Project Dike Plans (Continued)							
Plan 6	6	3	Fig 11		Note 2		
Plan 7	6	3	Fig 12		Note 3		
Plan 8	6	3	Fig 13		43-year avg	64	95
Plan 5A	6	—	Fig 14		43-year avg	102	92
Plan 5AO	6	—	Fig 14		43-year avg	Note 4	
Plan 5BO	4	—	Fig 15		43-year avg	Note 5	
					(150 days)		
45-ft Project Dike Plans							
Base	—	—	—	—	43-year avg	1617	—
Plan 8	6	3	Fig 13		43-year avg	140	91
Plan 8A	6	—	Fig 17		43-year avg	153	91
Plan 8AO	6	—	Fig 17		43-year avg	Note 4	
					(150 days)		
Plan 8BO	4	—	Fig 18		43-year avg	Note 5	
					(150 days)		

Note 1 Only steady flow tests were conducted with Plan 1. Comparison of results with comparable base tests indicated a reduction of 39 percent in channel deposition.
Note 2 Steady flow test to check sensitivity of channel deposition to further raising of Plan 5 dike crests.
Note 3 Steady flow test to check sensitivity of channel deposition to 200 ft extension of three right-bank dikes.
Note 4 Abbreviated 43-year-average-annual hydrograph run (150 high-flow days) to check effect of lowering crest of first two left-bank dikes by 5 ft. Little impact on channel depositon patterns.
Note 5 Abbreviated 43-year-average-annual hydrograph run (150 high-flow days) to check effect of eliminating lower two dikes on left bank. Singificant increase in channel deposition.

The initial dike plan for the 40-ft navigation channel (Plan 1) did not provide the desired reduction in maintenance dredging. Plans 2 through 6 represent a series of tests evaluating the impact of dike height on reduction of maintenance dredging. Each of these plans included six dikes on the left-descending bank and three dikes on the right-descending bank (Figure 11). Plan 7 investigated the sensitivity of channel deposition to a 200-ft extension of the three right-bank dikes. Plan 8 added 200-ft extensions to the three lower dikes on the left-descending bank and raised all dike crest elevations to +7 ft NGVD. Tests of Plan 5 with dike crests at 0 ft NGVD using the 43-year-average-annual hydrograph indicated the dredging requirement would be reduced by more than 90 percent. Plans involving higher crest elevations or dike length extensions produced only marginally larger decreases in required dredging. Consequently, limited additional testing was required to evaluate

modifications to Plan 5 which would minimize costs without significantly impacting its effectiveness. Plan 5A deleted the three dikes on the right-descending bank with no significant impact on navigation channel deposition. An abbreviated hydrograph test indicated only minor changes in channel deposition with the crest elevation of the upper two dikes on the left bank lowered 5 ft (Plan 5AO). The final test was an abbreviated hydrograph test with the lower two dikes on the left bank eliminated (Plan 5BO). Channel deposition was significantly increased with this dike plan. Results of this test series investigating dike plans for maintaining the 40-ft navigation project suggest Plan 5AO would provide significant reduction of maintenance dredging (43-year-average-annual hydorgraph) at minimum cost. The plan includes six dikes on the left-descending bank (Figure 14) with the crest elevation of the two upstream dikes at -5 ft NGVD and the crest elevation of the remaining four dikes at 0 ft NGVD. It should be noted that model estimates of reduction in maintenance dredging were "hydrograph sensitive." Limited testing with the 1990 hydrograph, a higher-energy hydrograph than the 43-year-average-annual hydrograph, suggested reductions in the 50-60 percent range rather than the 90-percent range achieved using the 43-year-average-annual hydrograph.

A short series of tests was conducted to evaluate dike plans for maintaining a 45-ft navigation channel. The starting point for these tests was Plan 8, equivalent to Plan 5 except the crest elevations of all dikes were raised from 0 ft NGVD to +7 ft NGVD and the three lower dikes on the left-descending bank included 200-ft extensions. The indicated reduction in channel deposition from the base condition was just over 90 percent for the 43-year-average-annual hydrograph. The optimization tests were the equivalent of the Plan 5 modification tests conducted for the 40-ft channel. Plan 8A estimated the effect of deleting the three dikes on the right-descending bank. Plan 8AO checked the effect of lowering the crest of the upper two dikes on the left bank and Plan 8BO estimated the impact of eliminating the lower two left-bank dikes. Results of this short test series investigating dike plans for maintaining a 45-ft navigation channel suggest Plan 8AO would provide significant reduction of maintenance dredging (43-year-average-annual hydrograph) at minimum cost. Although no tests were run with a higher peak discharge hydrograph, it is expected that the computed percent reduction in channel shoaling would have decreased substantially with such a hydrograph.

3 Movable-Bed Physical Model Investigation

Model Information

Description

The movable-bed model used for this study reproduced to a horizontal scale of 1:240 and a vertical scale of 1:200 the reach of the Mississippi River between miles 219.0 to 228.0 AHP, including the river channel and overbanks from the east to the west levees, except at the downstream end of the model where only about one-half of Manchac Point Bar was reproduced. This modeled area reproduced sufficient sections of the river upstream and downstream of Redeye Crossing and overbank areas to adequately study the problem. The scales were selected to produce a model channel width-to-depth ratio similar to several previous shallow-draft movable-bed model studies performed at WES. The model overbank above el +10[1] was molded in sand-cement mortar and steep channel banks below el +10 were molded using 3/4-in. crushed stone. The channel bottom was molded using crushed coal having a median diameter of 2 mm and a specific gravity of 1.30.

The model overbank configuration was obtained from maps in the 1973 to 1975 Mississippi River Hydrographic Survey book and U.S. Geological Survey maps, dated 1963, of the modeled area. Because of the limited overbank area, it was considered that the borrow pits and elevation changes on the overbank would not significantly affect the model results and would be time consuming and expensive to reproduce; therefore, the overbank was constructed flat at el +30. The channel bed configuration between miles 219 and 223 and between miles 225 and 228 was obtained from the 1983 to 1985 Mississippi River hydrographic survey, and the bed configuration from miles 223 to 225 was obtained from a 15 to 16 July 1982 dredging survey and included a 500-ft-wide dredge cut to el -40 along the normal alignment (azimuth 226° 25') dredged by the U.S. Army Engineer District, New Orleans (LMN). The Low Water Reference Plane (LWRP) in the area of

[1] All elevations herein are in feet referred to the Mississippi River Low Water Reference Plane (LWRP) datum (1974), unless otherwise stated.

Orleans (LMN). The Low Water Reference Plane (LWRP) in the area of Redeye Crossing is about el 2 ft NGVD, therefore, the dredge cut was at el -42 ft LWRP. Herein this bed configuration will be referred to as the September 1982 prototype survey (Plate 7).

Appurtenances

Water was supplied to the model by a 10-cfs centrifugal flow pump operating in a recirculating system and was measured with 12- by 6- and 6- by 3-in. venturi meters. A trough was provided along Manchac Point Bar to collect flow which passed over the point bar during high river stages. At that point, flow leaves the river channel in the prototype and reenters the channel downstream of the model limits. Water-surface elevations were controlled by slide-type gates at the downstream ends of the Mississippi River channel and the Manchac Point trough. Water-surface elevations were measured by piezometers located approximately 1 mile apart in the river channel and at the downstream end of the Manchac Point trough. Discharge over Manchac Point was obtained from results of the numerical model study and measured by calibration of the tailgate at the end of the trough. A graduated container was used to measure the crushed coal bed material introduced at the upstream end of the model. A sediment trap was provided at the downstream end of the Mississippi River channel where sediment extruded from the model could accumulate and be measured when desired. A carefully graded rail was installed along each side of the channel to support sheet metal templates used for molding the model bed prior to initiation of certain tests. These rails were also used to provide vertical control for surveying the model bed. A supplemental slope of 8 prototype feet was incorporated in both the river bed and overbank during construction of the model. Supplemental slope is required in a model of this type to adequately reproduce movement of the bed material.

Model verification

Before a movable-bed model is used to test the effectiveness of proposed improvement plans, its ability to reproduce conditions similar to those that can be expected in the prototype must be demonstrated. Complete similarity between the model and prototype results is seldom obtained because of the inherent distortions incorporated in the design and operation of the model. Because of these dissimilarities, the degree of reliability of this type of model cannot be fully established by mathematical analysis and must be based on model verification. Verification of the model involves the adjustment of various hydraulic forces, time scale, rate of introducing bed material, and model operating techniques until the model reproduces, with acceptable accuracy, the changes that occurred in the prototype during a given period. Various scale relationships and model operating procedures established during model verification are used in tests of improvement plans. The degree of similarity between the model and prototype data obtained during model verification is considered in the analysis of the model data.

Model verification tests were initiated with the model bed molded to conditions indicated by the September 1982 prototype survey (Plate 7). The model was operated using the blocked hydrograph from September 1982 to August 1983 (Plate 8) which represents flows that occurred in the prototype between this survey and a survey taken in August 1983 (Plate 9) as input for the 1983 to 1985 Mississippi River hydrographic survey. Herein, a repetition of a blocked hydrograph will be referred to as a run, and the August 1983 survey will be referred to as the August 1983 prototype survey. The model water-surface was controlled at Mile 224.0 (center of modeled reach) to reproduce the stages obtained from the September 1982 to August 1983 rating curve of the Baton Rouge, LA, gauge. At the end of each verification test, the model bed was surveyed and the resulting model bed configuration was compared with the bed configuration obtained in the August 1983 prototype survey. During the verification process, progressive changes were made in the discharge scale, supplemental slope, and the rate of bed material introduced until the model reproduced with a reasonable degree of accuracy the conditions indicated by the prototype survey of August 1983. For a detailed description of the verification process, see Franco (1978).[1]

Results obtained from the 2-D numerical model indicated that two consecutive runs of the hydrograph would produce a better verification of the model; therefore, the dredge cut was redredged to el -40 after an acceptable verification was obtained, and the model was subjected to a second run of the hydrograph. Verification Run 14 (Plate 10) shows the bed configuration obtained after the first run of the hydrograph and Verification Run 15 (Plate 11) shows the bed configuration obtained after the second run. Comparison of the results of the verification runs and the August 1983 prototype survey indicated that Redeye Crossing along the alignment of the dredge cut shoaled in Verification Run 14 about one-half as much as indicated in the prototype but was shoaled about the same as the August 1983 prototype survey after Verification Run 15. The model results indicated after Run 14 that the model would shoal somewhat higher than the prototype along the right descending bank forming a midbar between miles 225.5 and 227.0 and between the dredge cut and left river bank at Redeye Crossing. These areas continued to increase in size and elevation during Run 15. Comparison of Verification Run 14 with Verification Run 15 indicated that the amount of sediment deposited in the dredge cut increased about 33 percent after the second run. This indicates that after Redeye Crossing is dredged, the volume of sediment deposited in the crossing could increase. During operation of the model for verification, the dredge alignment was not dredged during the hydrograph although the prototype was dredged several times during the period from September 1982 to August 1983. Therefore, the volume of sediment deposited in Redeye Crossing could be higher than indicated in the model for one specific run.

---

[1] J. J. Franco. (1978). "Guidelines for the design, adjustment, and operation of models for the study of river sedimentation problems," Instruction Report H-78-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Tests and Results

Test procedure

After verification of the model, testing of all plans was conducted using three runs of the 43-year-average-annual hydrograph (Plate 1) and one run of the September 1982-August 1983 hydrograph (Plate 8). The 43-year-average-annual hydrograph was the 43-year-average-annual hydrograph developed by LMN for testing on the Old River Control Structures models less the flow diverted through the structures. The model water surface for the base tests was controlled the same as during verification. The water surface for all subsequent plan testing was controlled at the downstream end of the model to reproduce the stage obtained in the model during base tests for each flow. Because of the supplemental slope added to the model and the distortion of the discharge, the model water-surface profile can not be compared to the water-surface profile of the prototype. The 43-year-average-annual hydrograph is a relatively low-water year, while the 1982-1983 hydrograph is a relatively high-water year. After each run the model was surveyed and contoured, the navigation channel alignment was redredged to el -40, and the material was measured to determine the bed load deposited in the dredged channel. Each series of tests was started with the bed molded to the September 1982 prototype survey (Plate 7). Subsequent tests were started with the bed configuration obtained at the end of the preceding tests, except that the navigation channel cut was redredged. Only final results or significant changes produced by each plan are included in this report.

Base tests

Description. Before any proposed improvement plans were installed in the model, base tests were conducted to obtain a basis for comparing with the improvement plans. Base tests were conducted using the hydrograph sequence described earlier.

Results. Results with the 43-year-average-annual hydrograph (Runs 1 through 3) indicated that Redeye Crossing would shoal beginning from upstream and that the sediment would encroach into the dredge channel from the left. As with the verification tests, shoaling along the left bank occurred between the upstream end of the dredge cut and the left-descending bank and a midbar was formed in the upstream portion of the model. Both the left-bank shoaling and midbar were larger and somewhat higher with each successive run. Although the amount of material removed from the dredge cut remained about the same for all three runs, the location and maximum elevation of the deposit moved nearer the entrance of the crossing adjacent to the left-channel bank shoaling. The maximum elevation in the dredge cut was el -32, -29, and -28 for Runs 1, 2, and 3, respectively. Plate 12 shows the bed configuration obtained after Run 3. The volume of model material removed from the dredge cut for Runs 1, 2, and 3 was 0.75, 0.50, and 0.60 cu ft, respectively.

Results of Run 4 with the September 1982-August 1983 hydrograph, shown in Plate 13, indicated that the amount of material deposited in the dredged channel increased about 180 percent over Run 3 and the left-bank shoaling increased in size and elevation. After Run 4, the maximum elevation in the dredged channel was el -27 and the amount of model material removed was 1.1 cu ft.

Plan 8AO

Description. The LMN proposed dike plan for reducing shoaling at Redeye Crossing was tested in the 2-D numerical sedimentation model to evaluate and refine the proposed plan before testing in the physical model. The numerical model indicated that the three dikes proposed on the right-descending bank were ineffective and that only the six dikes proposed on the left-descending bank would be required to maintain an el -40 channel, but higher dike crest elevations than proposed would be required. The results of the numerical model indicated that the same dike plan with the dike crests raised an additional 7 ft would maintain a 45-ft-depth channel with minimum maintenance dredging required. Plan 8AO is the refined plan developed in the numerical model for a 45-ft navigation channel. Plan 8AO was selected for testing in the physical model except the channel was dredged to el -40 for the 40-ft navigation project, so a direct comparison could be made to the base tests runs completed in the physical model. Plan 8AO consists of six spur dikes tied to the left bank near the upstream end of Redeye Crossing. The dike field for Plan 8AO, shown in Plates 14 and 15, was as follows:

Dike No.	Location Mi AHP	Length ft	Crest Elev NGVD, ft	Crest Elev LWRP, ft
1	224.4 L	380	+2	0
2	224.3 L	530	+2	0
3	224.1 L	860	+7	+5
4	223.7 L	970	+7	+5
5	223.4 L	1,270	+7	+5
6	223.1 L	1,300	+7	+5

Plan 8AO was tested using the same sequence of hydrographs used for the base tests.

Results. The bed configuration obtained after the third run of the 43-year-average-annual hydrograph is shown in Plate 16, and the bed after the Sep 82-Aug 83 hydrograph is shown in Plate 17. Results of dredging the channel to el -40 after each run indicated the amount of sediment deposited in the dredge cut, although not eliminated, would be reduced for the 43-year-average-annual hydrograph. Results with the Sep 82-Aug 83 hydrograph indicated that the deposited material would be reduced but not as much as with the lower

water-year 43-year-average-annual hydrograph. As during base tests, the higher water-year (Sep 82-Aug 83) hydrograph drastically increased the amount of sediment deposited in the 500-ft-wide dredge cut channel. The highest elevations in the dredge channel were about the same as in the base tests and were always near the ends of dikes 1 and 2. Shoaling in the crossing was concentrated near the upstream end of the crossing with Plan 8AO in place. The maximum elevations in the crossing were -32, -29, -27, and -25 for runs 1 through 4, respectively. The following is a comparison of the model bed material removed from the dredge cut after each run for conditions with and without the dike field:

		Model Bed Material Removed		
Run	Hydrograph Used	Base Test cu ft	Plan 8AO cu ft	Percent Decrease
1	43-year average	0.8	0.4	50%
2	43-year average	0.5	0.2	60%
3	43-year average	0.6	0.2	67%
4	Sep 82 - Aug 83	1.1	0.8	27%

These test results indicate that, although the amount of material deposited in Redeye Crossing would be reduced, dredging would be required to maintain a 40-ft navigation channel, with the dredging effort being more extensive during high-water years.

Plan 8AO-45

Description. After the completion of tests of Plan 8AO, Plan 8AO-45 was tested to provide some indication of the dredging requirements for a 45-ft navigation channel. The tests were initiated with the bed configuration obtained after Plan 8AO run 4, except the dredge cut was dredged to el-45. An additional 1.1 cu ft of coal was removed from the dredge channel, therefore the total material removed was 1.9 cu ft for a 45-ft channel. The model was subjected to one run of the Sep 82-Aug 83 hydrograph.

Results. The bed configuration following this run, shown in Plate 18, indicated that the amount of material deposited in the crossing increased dramatically. The amount of material removed from the dredge channel was 3.3 cu ft. The shoaling in the dredge channel occurred on top of the ends of dikes 1 and 2 with a shoal of about 20 ft occurring at the ends of dikes 3, 4, and 5. The maximum elevation in the dredge channel was el -24. Some deepening of the channel occurred at the downstream end. The bed configuration obtained after this test is about the same as the configuration obtained after base tests Run 4 (Plate 13) without the dikes, except for a large increase of deposited material in the area of dikes 1 through 3. This

comparison indicates the dikes had very little effect on the shoaling pattern in Redeye Crossing.

Plan 8AO channel shift

Description. The results of Plan 8AO-45 indicated some scour would occur at the downstream end of the dredge cut, and the numerical model indicated the channel would shift toward the ends of the dikes; therefore, an attempt to shift the channel using an extended period of a high flow was undertaken. The higher velocities and straightened current alignment of the prolonged higher stage should indicate any tendency for the channel shift toward the dike field. The model, with the bed configuration obtained at the end of the previous test and the dredged channel redredged to el -45, was subjected to 16 hr (160 prototype days) of the Sep 82-Aug 83 hydrograph peak stage (43-ft stage).

Results. The bed configuration at the end of the 16-hr run is shown in Plate 19. The results indicated that the bar formed at the ends of dikes 3 and 4 in the previous tests would continue to increase in elevation and size. The bar extended downstream of dike 6 and riverward about 800 ft from the end of dike 6. Almost the entire dike field downstream of dike 3 was shoaled above el +20. The dredged channel was shoaled between dikes 1 and 4, with 4.6 cu ft of material deposited in the channel. These results indicate the channel has no tendency to shift toward the dike field.

Plan 8A1

Description. The results of the previous tests with the Plan 8A0 dike field and the fact that most of the sediment is moved during higher flows with the lower flows not having enough energy to scour the crossing, indicated that the plan dikes do not constrict the channel width enough to significantly increase the sediment carrying capacity of the falling river flows. The test of Plan 8A1 was conducted to obtain an indication if higher dikes could maintain an acceptable crossing channel. Since the bar at the ends of dikes 3 through 6, after Plan 8A0-Channel Shift, was near el +20 or higher, it was felt that the bar would adequately represent longer dikes near midbank or higher elevations. Plan 8A1 was subjected to one run of the Sep 82-Aug 83 hydrograph on the bed configuration obtained at the end of Plan 8AO-Channel Shift with the dredge channel redredged to el -45.

Results. The bed configuration obtained after the run of the hydrograph is shown in Plate 20. The results indicate the bed configuration was unchanged except in the crossing where the bar at dikes 3 through 6 remained about the same shape but was slightly higher. The dredge channel downstream of dike 4 was not filled, but upstream of dike 4 the channel shoaled about 10 to 15 ft with the highest elevation being -28 ft. To redredge the channel, 4.8 cu ft of material was removed from the model. These results indicate

longer and higher dikes would be required to maintain a deep-draft navigation channel in Redeye Crossing on the falling side of the hydrograph.

Discussion of Results

Interpretation of Physical Movable-bed Model Results

The limitations of the model in reproducing all the factors affecting developments in the reach and the differences between the model and prototype indicated by results of verification tests must be considered in the evaluation of the model results. It should be considered that the model does not reproduce the movement of material in suspension, and that the bank lines are fixed, with no attempt to reproduce the degree of erodibility of the banks. Also to be considered is the 43-year-average-annual and September 1982-August 1983 hydrographs used for testing of plans, which could be considerably different from what actually occurs in the river in the future, and the fact that the model surveys were always made during low-water periods. In spite of these limitations, verification of the model was sufficient to indicate trends that can be expected under the conditions imposed for each plan tested and the relative effectiveness of each plan. Results of improvement plans should be based on only the changes caused by these plans compared with the results reproduced in the model during base tests. Dredging quantities are presented only to show relative effectiveness of the plans tested and cannot be used to determine the quantity of material to be dredged in the prototype for any particular plan.

Specific test results

The following tabulation summarizes results from the physical model tests.

Time constraints (Chapter 1) limited physical model testing to one dike plan. The plan selected was Plan 8AO from the numerical model study, the dike plan with crest elevations designed for maintaing a 45-ft project. In the physical model, the plan was tested with both a 40-ft channel project and a 45-ft channel project. In the 40-ft project, the dike plan reduced channel deposition compared to base tests by about 60 percent in tests with the 43-year-average-annual hydrograph and by about 27 percent with the Sep 82-Aug 93 hydrograph (peak flow almost double peak flow of 43-yr-average-annual hydrograph). Model results indicated that the maximum bed elevation in the navigation channel through the crossing was about the same with the dikes in place as without the dikes, but the shoal did not extend as far into the crossing with the dikes in place.

The one test of Plan 8AO dikes with a 45-ft channel (Plan 8AO-45) was conducted using the Sep 82-Aug 83 hydrograph and the bed configuration existing at the conclusion of a similar test for a 40-ft channel. Model results

Test Condition	Number of Dike: Left Bank	Number of Dike: Right Bank	Dike Layout	Dike Elev	Hydrograph	Channel Infill ft^3	Percent Reduction from Base
40-ft Project							
Base	—	—	—	—	43-yr avg-Run 1	0.8	—
					" " -Run 2	0.5	—
					" " -Run 3	0.6	—
					Sep 82-Avg 83	1.1	—
Plan 8AO	6	—	PL 14	PL 15	43-yr avg-Run 1	0.4	50
					" " -Run 2	0.2	60
					" " -Run 2	0.2	67
					Sep 82-Aug 83	0.8	27
45-ft Project							
Plan 8AO-45	6	—	PL 14	PL 15	Sep 82-Aug 83	3.3	[1]
Plan 8AO-45	6	—	"	"	Sep 82-Aug 83	4.6	
(Channel Shift)					Peak Flow, 160 days		Note 1
Plan 8A1	6	—	Bed after Plan 8AO-45 (Channel Shift) except el -45 channel redredged		Sep 82-Aug 83	4.8	Note 2

[1] No base test was run for a 45-ft channel. A 45-ft channel was dredged in the bed configuration existing at the end of the Sep 82-Aug 83 hydrograph run of Plan 8AO and a 40-ft project. Total excavation required to establish a 45-ft channel was 1.9 ft^3.

Note 1 Extended period of high flow run to determine if model showed any tendency for channel to move toward dike field.

Note 2 Test conducted with high bar elevations existing at end of Plan 8AO-45 (Channel Shift) run to determine if additional channel constraction would tend to maintain acceptable crossing channel.

indicated the dikes had little effect on final bed elevations in Redeye Crossing and thus, the required dredging to restablish the navigation channel was 3.3 ft^3, i.e., much higher for the 45-ft channel than for the 40-ft channel. However, no direct comment can be made concerning the benefit of dikes, since no base test was run to estimate channel shoaling in a 45-ft channel without dikes.

A special steady high flow test (Plan 8AO-Channel Shift) lasting the equivalent of 160 days was run to determine if the physical model would indicate any tendency for the navigation channel to rotate toward the dike field. Some numerical model results had indicated a tendency for such migration. Physical model results showed continued building of the bar off the ends

of the lower dikes and no tendency of the navigation channel to shift toward the dikes.

Since physical model results suggested that significant additional constriction of the channel would be required to materially increase the sediment carrying capacity of the falling river flows, one additional test was conducted. This test (Plan 8A1) used the major bar growth (elevation and extent) resulting form the Plan 8AO-Channel Shift test as a crude representation of the effect of longer, higher dikes. The test was run with the Sep 82-Aug 83 hydrograph. The bed configuration did not change significantly but the upstream portion of the crossing channel shoaled about 10 to 15 ft with the highest elevation about -28 ft. Redredging the 45-ft channel required removal of 4.8 cu ft of material, suggesting longer and higher dikes would be required to maintain a deep-draft navigation channel in Redeye Crossing on the falling side of the hydrograph.

4 Analysis of Results and Conclusions

Analysis of Study Results

The sedimentation study plan included use of both numerical and physical models. The numerical models were to be used during the period required for physical model construction and verification to determine an approximate contraction width and provide preliminary guidance on dike spacing and location. This screening process was adopted to minimize the testing program required in the physical movable-bed model. Although adjusted somewhat by changed study requirements in this case (Chapter 1), this basic approach is still considered valid. After initial calculations using the 1-D numerical sedimentation model (TABS-1) to estimate the required contraction and provide boundary conditions, the 2-D numerical modeling system (TABS-2) was used to test a number of alternative dike plans at Redeye Crossing. Eleven different dike plans were tested with the 40-ft navigation channel to provide insight concerning the number of dikes and their crest elevation, location, and length. Four additional dike plans were tested to provide similar information for a 45-ft navigation channel. The optimum plans based on numerical model results, Plan 5AO for the 40-ft channel (Figure 14 and Chapter 2, paragraph entitled "Dike crest elevations and lengths") and Plan 8AO for the 45-ft channel (Figure 17 and Chapter 2, paragraph entitled "Dike crest elevations and lengths"), differed in dike crest elevation with the dikes for the 45-ft channel being 7 ft higher than those for the 40-ft channel and in the length of the three downstream-most left-bank dikes being extended by 200 ft for the 45-ft channel. In the interest of time, all physical movable-bed model tests were conducted with dike Plan 8AO whether the navigation channel was dredged to el -40 or -45 ft.

Although test schedule requirements resulted in physical model tests which were not directly comparable to previous numerical model tests, there are sufficient similarities between 40-ft channel tests in the two models to make some qualitative comparisons. That is not true for the 45-ft channel tests since no base test without dikes was conducted in the physical model. However, for the 40-ft channel, most general tendencies from the two modeling approaches were consistent. Both models indicated that dikes would reduce

shoaling in the crossing channel but that dike elevations would have to be substantially higher than those originally proposed by LMN and based on the conveyance procedure developed from prototype experience with dike fields and naturally maintaining crossings in shallow-draft channels. However, numerical model estimates of dike effectiveness were much higher than physical model estimates (discussed below). The numerical model results indicated that dikes proposed on the right descending bank were generally ineffective and, although no specific tests were conducted in the physical model with dikes on the right bank, physical model observations supported the indication that they would be ineffective. Results from both models indicated that the volume of channel shoaling with the dike plans is highly dependent on the flow hydrograph, with the effectiveness decreasing as the magnitude of the flow hydrograph increases. Two significant differences also were evident from analysis of numerical and physical model results: (a) a tendency for the crossing channel to shift toward the dike field in the numerical model results that was not exhibited in physical model results and (b) numerical model estimates of shoaling reduction were substantially higher than estimates based on physical model results. The two situations are discussed in the following paragraphs.

Tendency of channel to rotate toward dike field

The numerical sediment transport computations with the TABS-2 modeling system indicated a tendency for the downstream end of the crossing channel to migrate toward the dike field leading to a rotation of the navigation channel alignment through the crossing. Later tests of dike plans in the physical movable-bed model did not demonstrate that tendency. This leads to the conclusion that the numerical model tendency for the channel to rotate toward the dike field is probably an artifact of its inability to capture the three-dimensional (3-D) character (rotational flow) of flow in the bendway just downstream of Redeye Crossing. The depth-averaged numerical model has a tendency to move the strength of flow to the inside of the bend resulting in poor lateral distribution of flow in the bendway. This known weakness of the depth-averaged model also prevents the model from properly building the point bar which also may contribute to the computed channel migration tendency. These factors lead to the conclusion that installation of the tested dike field should not significantly impact present navigation channel alignment at Redeye Crossing.

Effectiveness of dike plans in reducing channel shoaling

As noted earlier, while the general tendencies of the numerical and physical model results for the 40-ft channel tests were consistent, there were substantial differences in the estimates of shoaling reductions achieved by the tested dike plans. The physical model test results indicated less benefit from dike field installation than the numerical model had indicated. The discussion of these differences should begin with the acknowledgment that both

approaches are approximations of a very complex physical process, and their validity is impacted by a variety of factors. The following paragraphs discussing each approach are an attempt to put some of those factors into perspective.

Numerical model (TABS-2). The basic limitation of the 2-D model in not computing the rotational flow in the bendway is equally relevant to the computation of shoaling estimates. The approximation of flow conditions through the upstream portion of the crossing should be realistic with some loss of validity as the bendway is approached, and the inability of the model to properly reproduce the point bar development becomes more important.

Another factor influencing accuracy of predictions is model adjustment and associated data limitations. The Redeye Crossing Reach has a very flat energy slope, and limited data were available for use in hydrodynamic model adjustment. Stage data from the nearest upstream and downstream gauges were used to adjust water-surface profiles in the 1-D model and then the 2-D model was adjusted to stage-discharge relationships developed from the 1-D calculations. Very limited prototype velocity data were available, primarily data collected to help establish the proper flow distribution across Manchac Point during high flows. Overall, hydrodynamic model adjustment was accomplished with much less field data than is typically available for 2-D model verification. Nevertheless, in general, computed stage and velocity information appeared consistent and reasonable.

As is typical of many sediment transport studies, available data for verification were limited. Sediment inflow concentrations for the 2-D study were taken from the 1-D model which was adjusted using measured concentrations at selected Mississippi River gauge locations. Bed sediment data were available from the dredged crossing channel and, from these data, a medium sand (0.25 mm) was selected as the single grain size for transport. Other input data and coefficients were given in Chapter 2 of this report. The model was adjusted for the 43-year-average-annual hydrograph to yield a channel shoaling volume similar to the average maintenance dredging reported in years when peak daily discharge did not exceed 1,000,000 cfs. Time constraints did not permit significant sensitivity tests with higher energy hydrographs during the adjustment process. One problem encountered during the adjustment process was the tendency to erode the point bar downstream of Redeye Crossing. Since the bar was downstream of the study area and time was critical, the bar was hardened (i.e., no erosion allowed). The question remains as to whether this trend was an artifact of the depth-averaged model's inability to correctly compute the lateral flow distribution in the bendway or some other factor.

While it is not possible to quantify the accuracy of the numerical model results, the cumulative effect of these factors is probably optimistic estimates of shoaling reduction from dike plans. As noted earlier, use of the depth-averaged numerical model was proposed as a screening tool to limit the test program in the physical model. Circumstances which developed resulted in its

use for developing design refinements in spite of the inherent limitations of the 2-D code in modeling the important 3-D aspects of the Redeye Crossing Reach. Some additional insight concerning dike field effectiveness could probably have been developed through additional sensitivity testing, etc., but a fully 3-D flow/sediment transport code is needed for substantial improvement in numerical model shoaling predictions at Redeye Crossing.

Physical movable-bed model. The ability of a physical movable-bed model to reproduce conditions similar to those that can be expected in the prototype is highly dependent upon the success of model verification efforts. Although rooted in science, physical movable-bed modeling is largely an art due to the inability to scale all pertinent phenomena. It is an art which has been highly developed and successfully applied for study of shallow-draft problem areas over the past 40 to 50 years. However, the Redeye Crossing study is the first WES application of this movable-bed model technology to a deep-draft, open river problem area. Consequently, the verification process was complicated by the necessity to extrapolate techniques and procedures which worked in shallow-draft situations to similar techniques and procedures which would work in the deep-draft environment. (The positive side of that situation is that the flatter portion of that learning curve is now history and future deep draft studies will benefit.)

The Redeye Crossing Reach also presented unique problems. The reach includes a relatively short radius, large curvature bend just downstream of the crossing and available field data indicated the elevation of the point bar did not follow the "normal" tendency of alternating point bars to be very dependent on the flow hydrograph. These factors coupled with the necessity of finding the right distortion for discharge and bed slope to encourage appropriate movement of the model bed material made the verification process difficult and time consuming.

As noted earlier, the urgency in the spring of 1991 for guidance on dike design to help finalize a plan for construction in 1991 led to termination of verification refinements and an abbreviated physical model testing program. As with the numerical model results, it is not possible to quantify the accuracy of the movable-bed physical model results. However, the judgment is that physical model results are qualitative and that the estimates of shoaling reduction from dike plans are conservative. Had circumstances on the physical movable-bed model permitted additional refinement during the verification, then the test results could be extrapolated to a more precise prediction of the plan results on the channel development. In other words, the more refined the verification, the more comfortable one would feel that any particular model plan result would be predicting the effect of that plan in the prototype.

Conclusions

The Redeye Crossing Reach is a very complex physical system with the

crossing followed immediately by a relatively short radius, large curvature bend. Field data indicated the added complexity that the elevation of the point bar in this bend does not follow the normal tendency of alternating point bars to be very dependent on the flow hydrograph.

The study plan concept of using both numerical models and a physical movable-bed model was a useful approach. The numerical model was used to screen several dike plans to select the most promising plans for more extensive testing with the numerical model and in the physical model.

For the 40-ft channel tests where test conditions were similar enough for qualitative comparison, most general tendencies from the two modeling approaches were consistent.

Both numerical and physical models documented that the effectiveness of dike plans in reducing channel shoaling is hydrograph dependent with the effectiveness decreasing as the magnitude of the flow hydrograph increases.

Both numerical and physical models indicated that, for the 40-ft project, dikes would reduce shoaling in the crossing channel. The models also indicated that dike crest elevations would have to be substantially higher than those originally proposed based on the conveyance procedure developed from prototype experience with dike fields and naturally maintaining crossings in shallow-draft channels. However, numerical model estimates of dike effectiveness were much higher than physical model estimates.

40-ft channel. Relative to model results of tests for the 40-ft channel, the following conclusions are presented:

a. Numerical model test results indicated Dike Plan 5AO (6 dikes on left-descending bank, crest elevations -5, -5, 0, 0, 0 upstream (U/S) to downstream (D/S)) would reduce channel shoaling by about 90 percent for the 43-yr-average-annual hydrograph and 50 to 60 percent for the 1990 hydrograph.

b. Physical movable-bed model tests results indicated Dike Plan 8AO (6 dikes on left descending bank, crest elevations 2, 2, 7, 7, 7, 7 U/S to D/S) would reduce channel shoaling by about 60 percent for the 43-yr-average-annual hydrograph and about 27 percent for the Sep 82-Aug 83 hydrograph.

c. Both projections are estimates with current judgment that numerical model estimates are optimistic and physical model estimates are conservative.

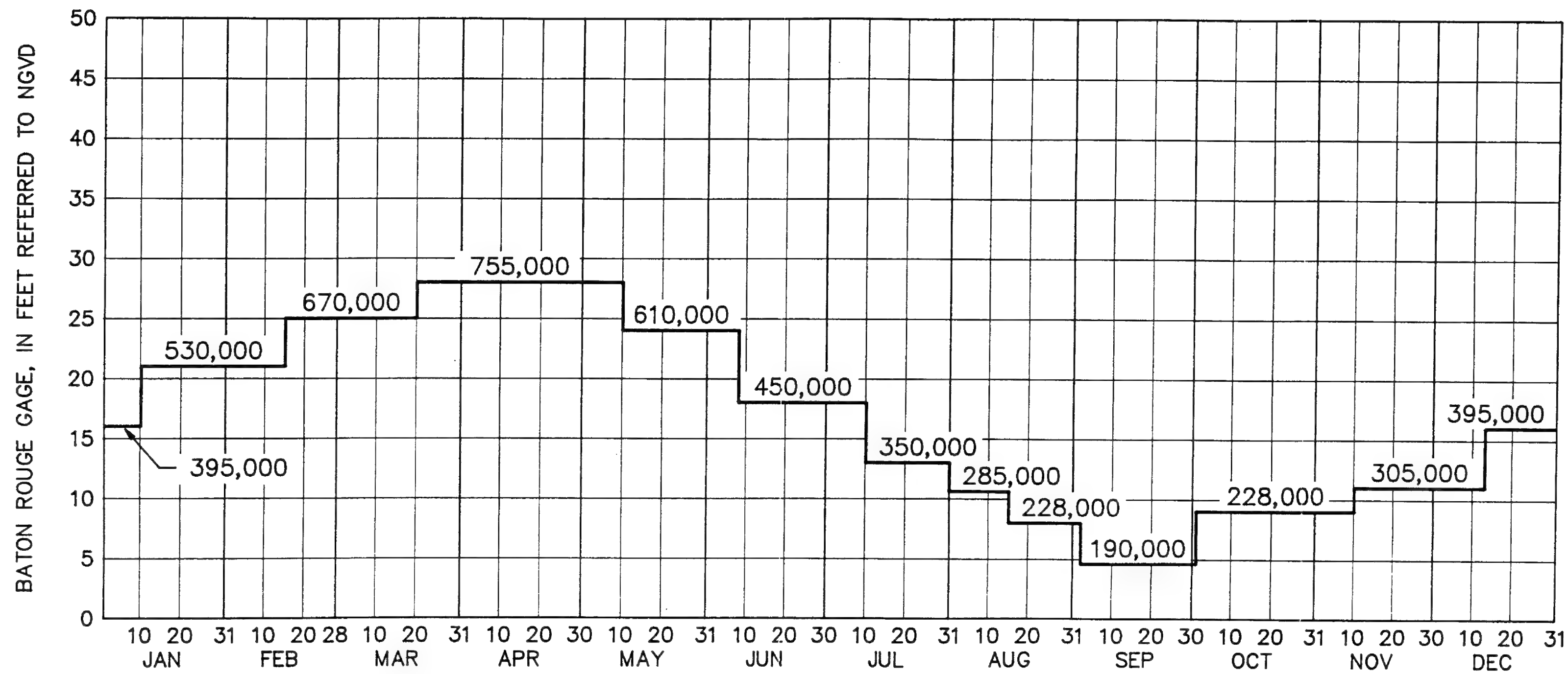
45-ft channel. Relative to model results of tests for the 45-ft channel, the following conclusions are presented:

a. Numerical model test results indicated Dike Plan 8AO (described above) would reduce channel shoaling by about 90 percent for the 43-year-average-annual hydrograph. No test was run with a higher flow hydrograph.

b. Physical movable-bed model tests were conducted with Dike Plan 8AO but, since no base test was run, a percent reduction in channel shoaling can not be presented. The volume of channel shoaling with Plan 8AO and the Sep 82-Aug 83 hydrograph was much higher than the similar test with the 40-ft channel.

c. Judgment is that numerical model estimates are optimistic and physical model results are conservative.

Installation of the tested dike field should not significantly impact present navigation channel alignment at Redeye Crossing. The numerical model tendency for the downstream end of the crossing to rotate toward the dike field is probably an artifact of its inability to capture the 3-D character (rotational flow) of flow in the bendway just downstream of Redeye Crossing. Physical model test results did not indicate a tendency for the alignment to rotate.



NOTE: VALUES SHOWN ON HYDROGRAPH ARE DISCHARGE IN C.F.S.

STAGES ARE AT BATON ROUGE, LA AND LWRP = 2 FEET NGVD

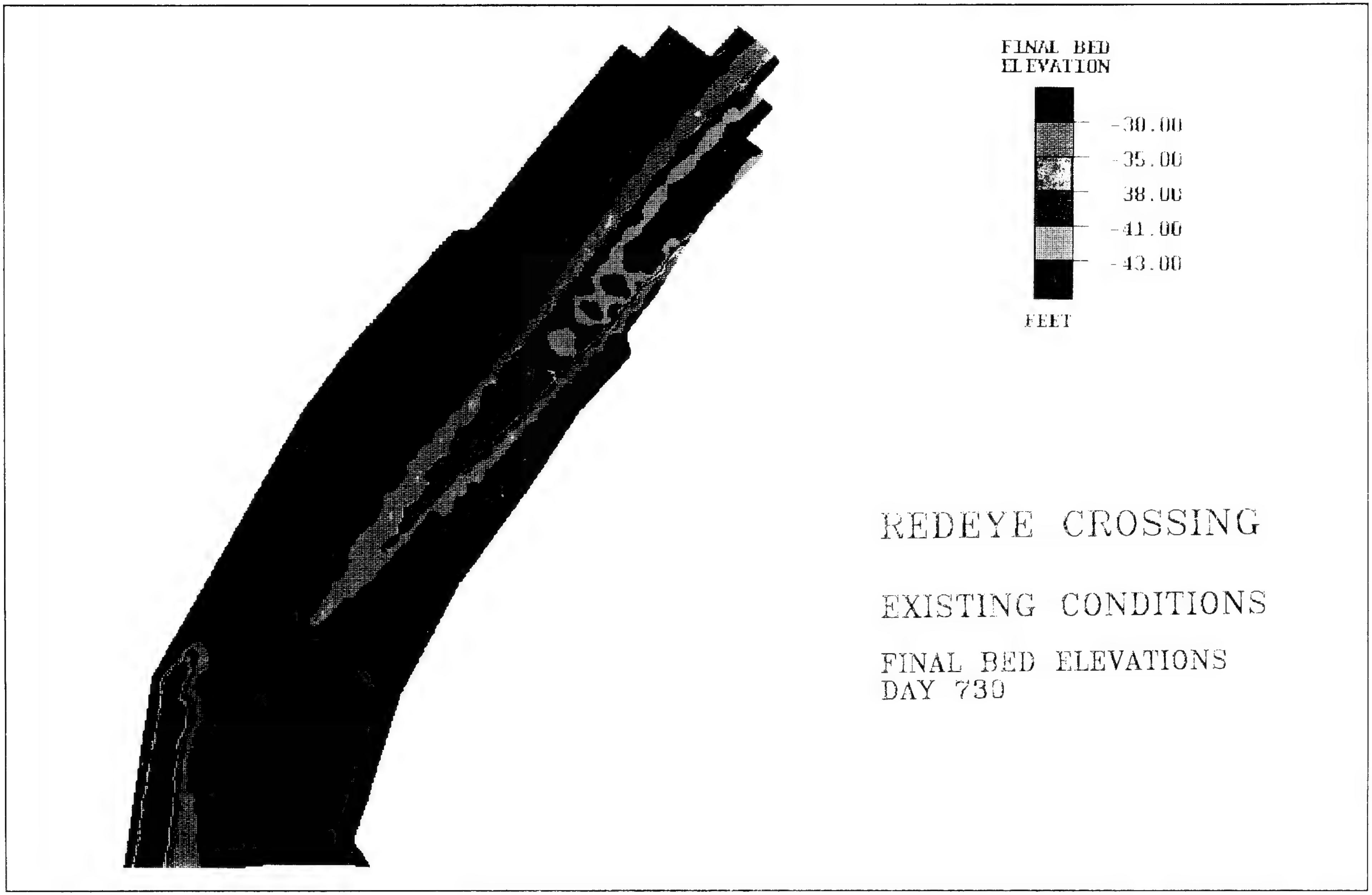
MODEL STUDY OF
REDEYE CROSSING REACH, MISSISSIPPI RIVER

43 - YEAR AVERAGE ANNUAL HYDROGRAPH AT BATON ROUGE, LA

REDEYE CROSSING

EXISTING CONDITIONS

FINAL BED ELEVATIONS
DAY 365


FINAL BED ELEVATION
-30.00
-35.00
38.00
-41.00
-43.00
FEET
REDEYE CROSSING
EXISTING CONDITIONS
FINAL BED ELEVATIONS
DAY 730

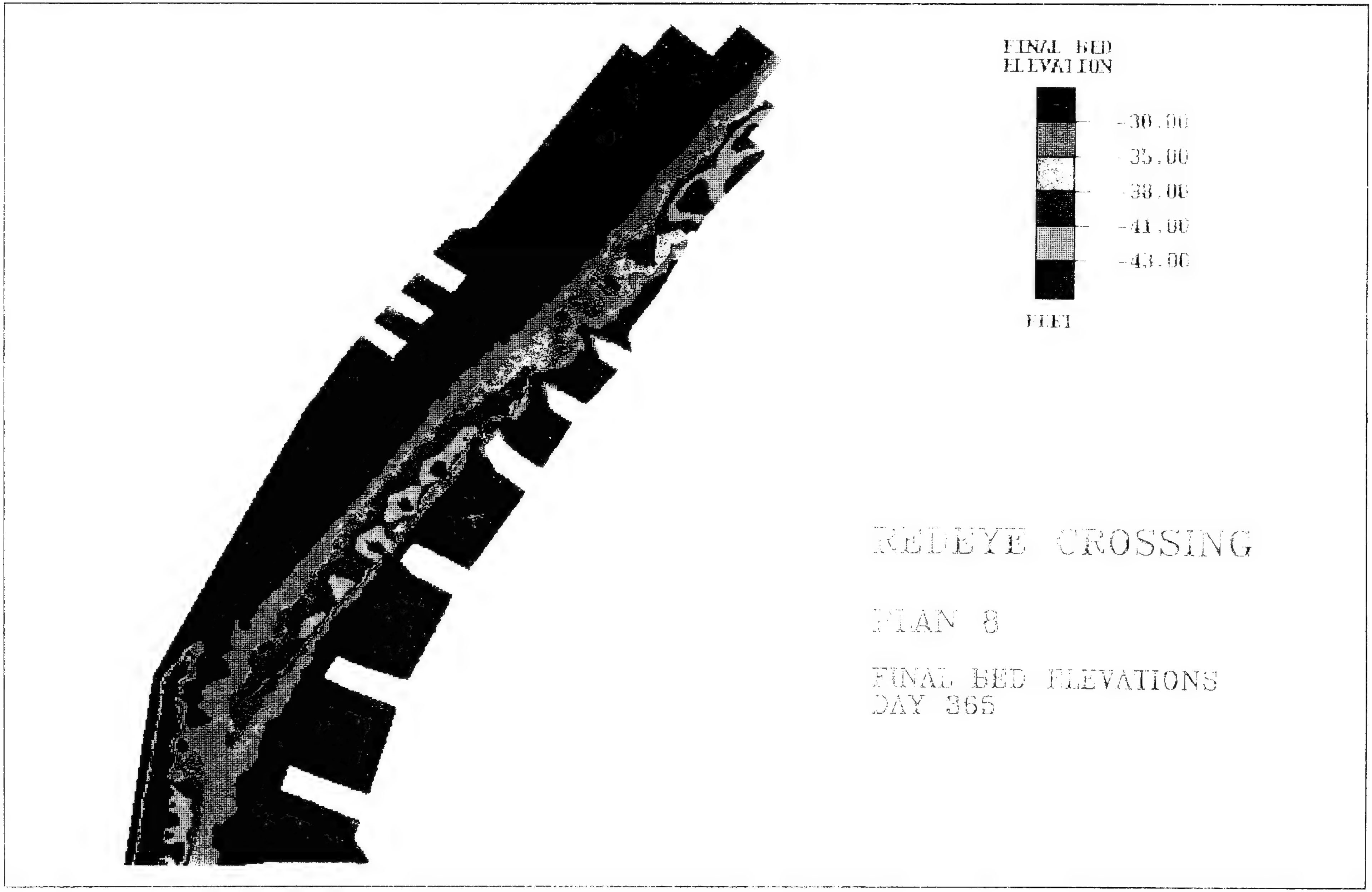
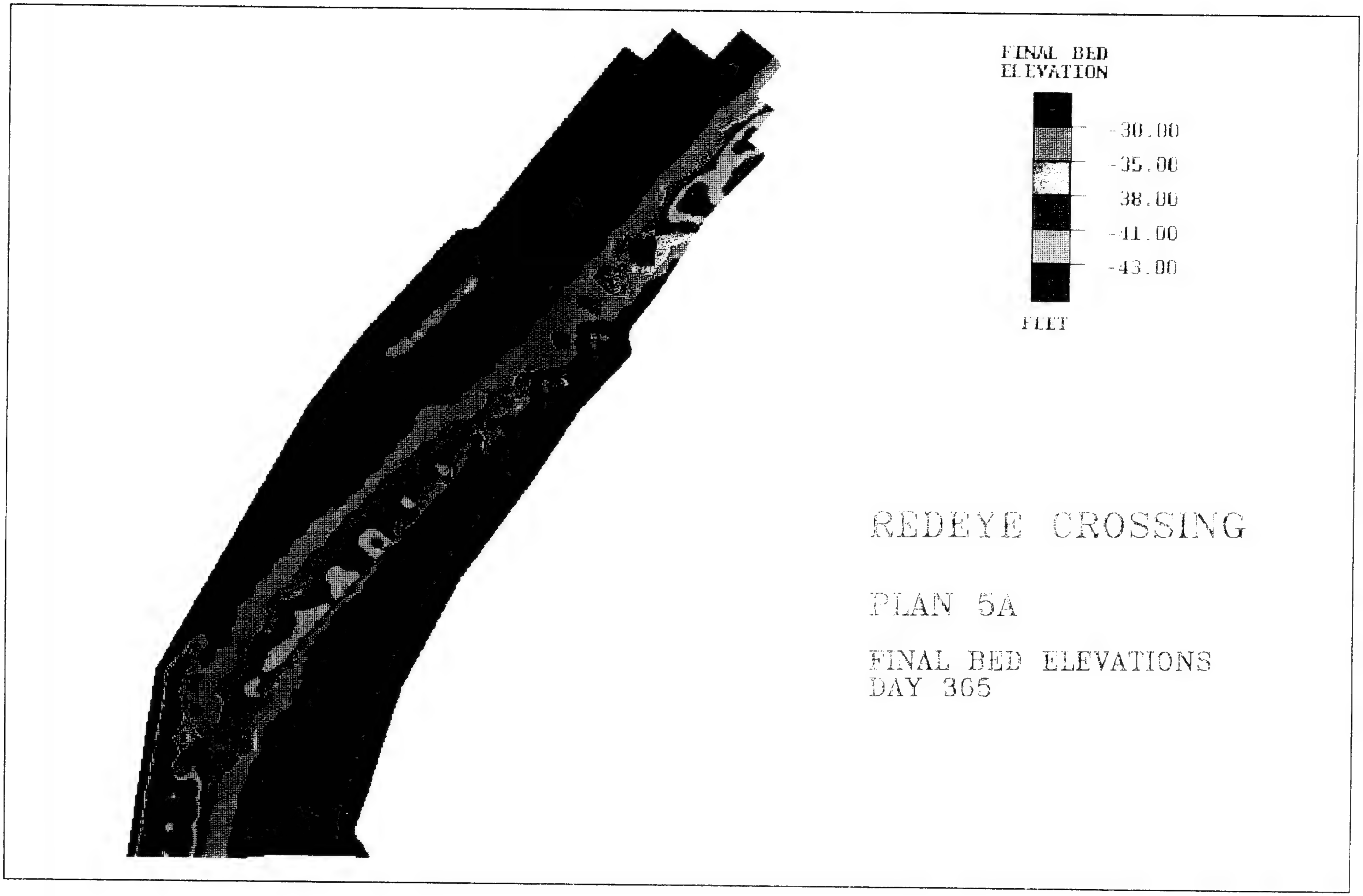
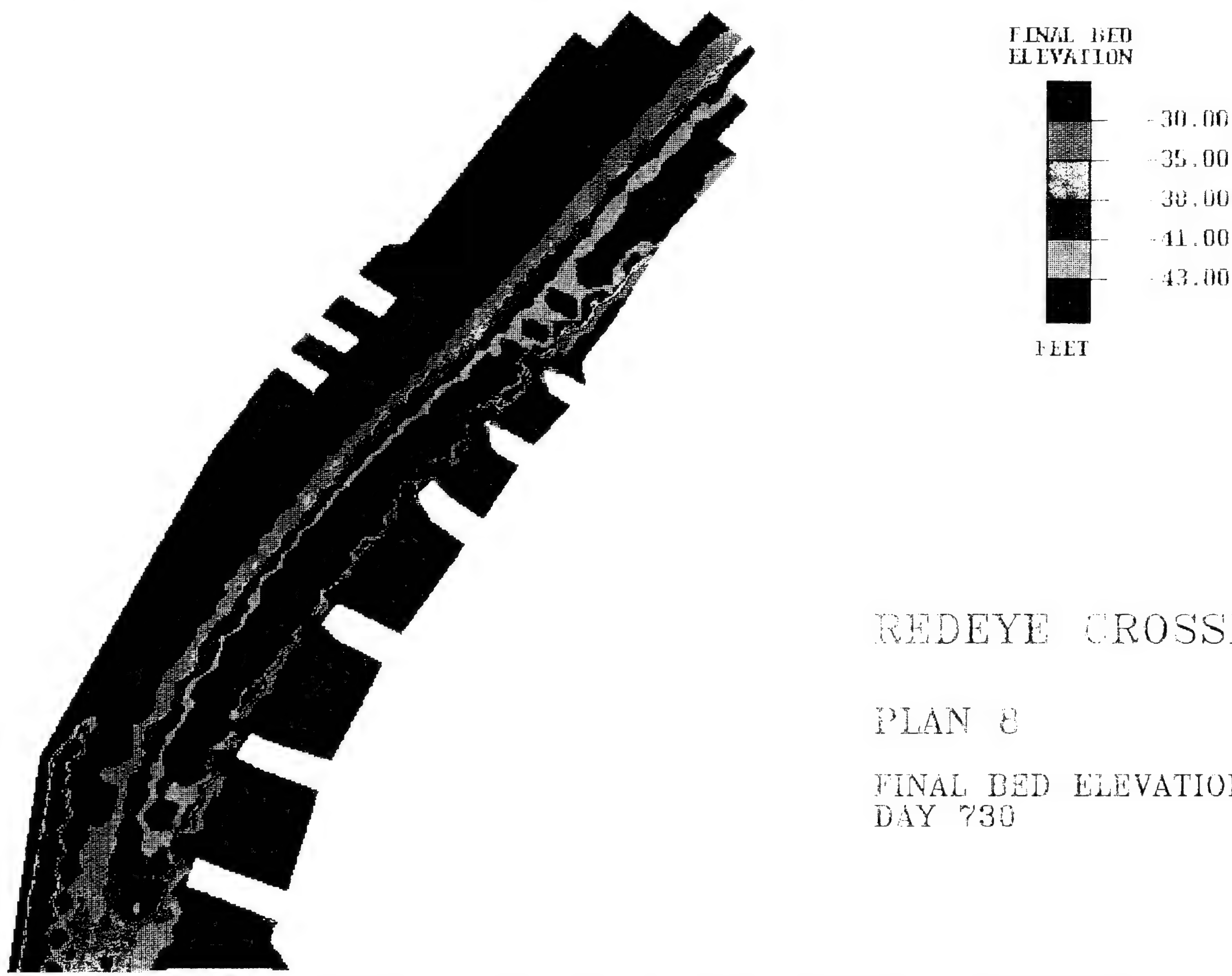

FINAL BED
ELEVATION
-30.00
-35.00
-38.00
-41.00
-43.00
FEET
REDEYE CROSSING
PLAN 8
FINAL BED ELEVATIONS
DAY 365

Plate 4

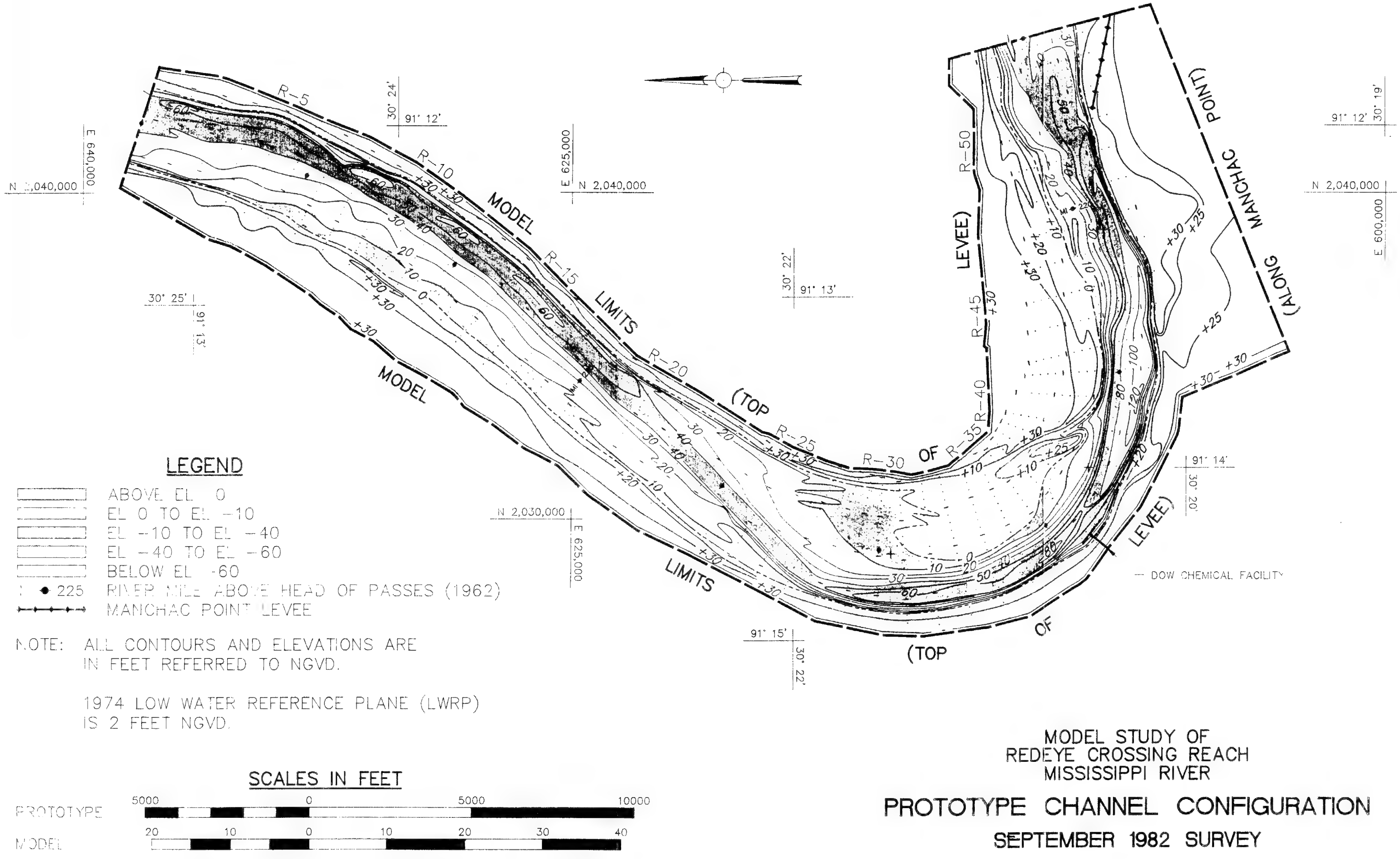

FINAL BED
ELEVATION
-30.00
-35.00
38.00
-41.00
-43.00
FEET
REDEYE CROSSING
PLAN 5A
FINAL BED ELEVATIONS
DAY 365



REDEYE CROSSING

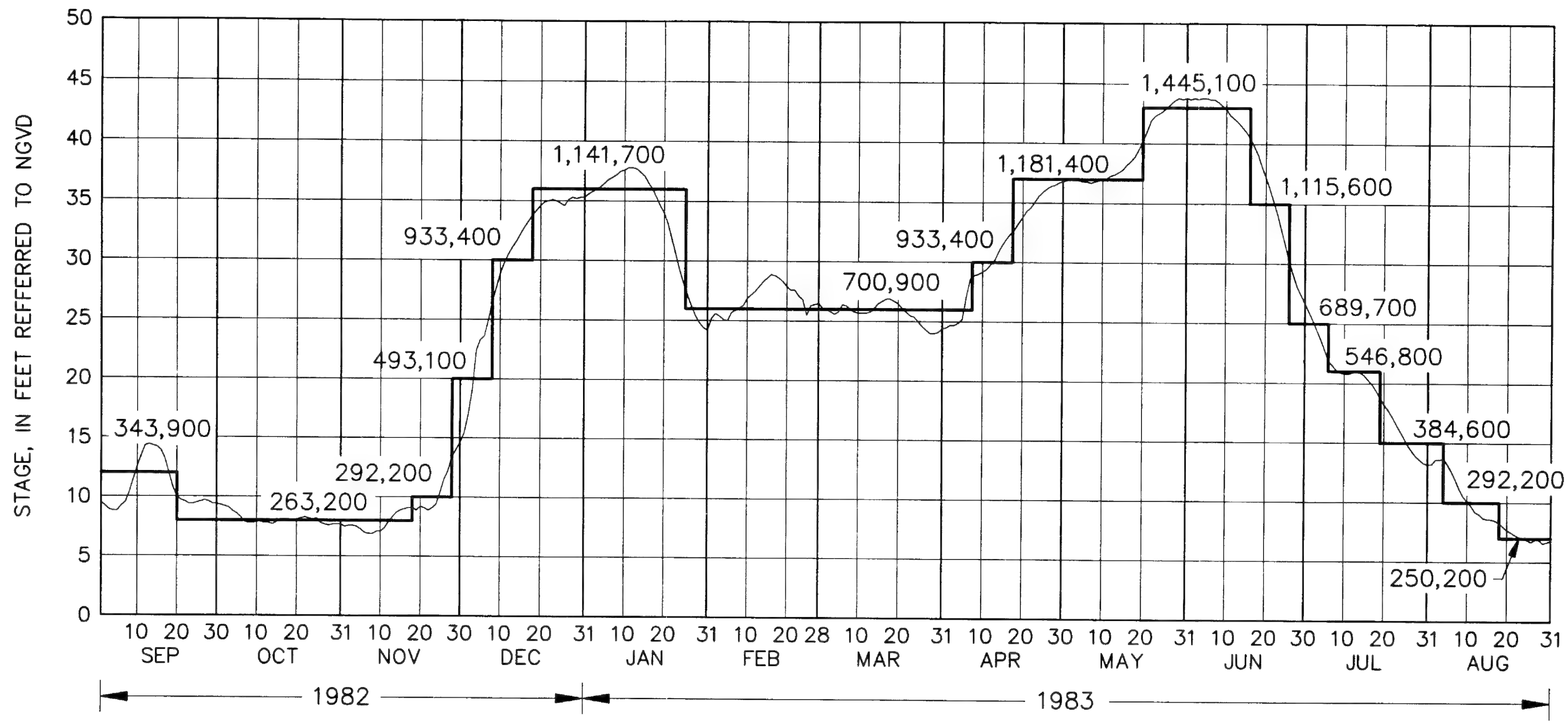
PLAN 8

FINAL BED ELEVATIONS
DAY 730



MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER

PROTOTYPE CHANNEL CONFIGURATION
SEPTEMBER 1982 SURVEY

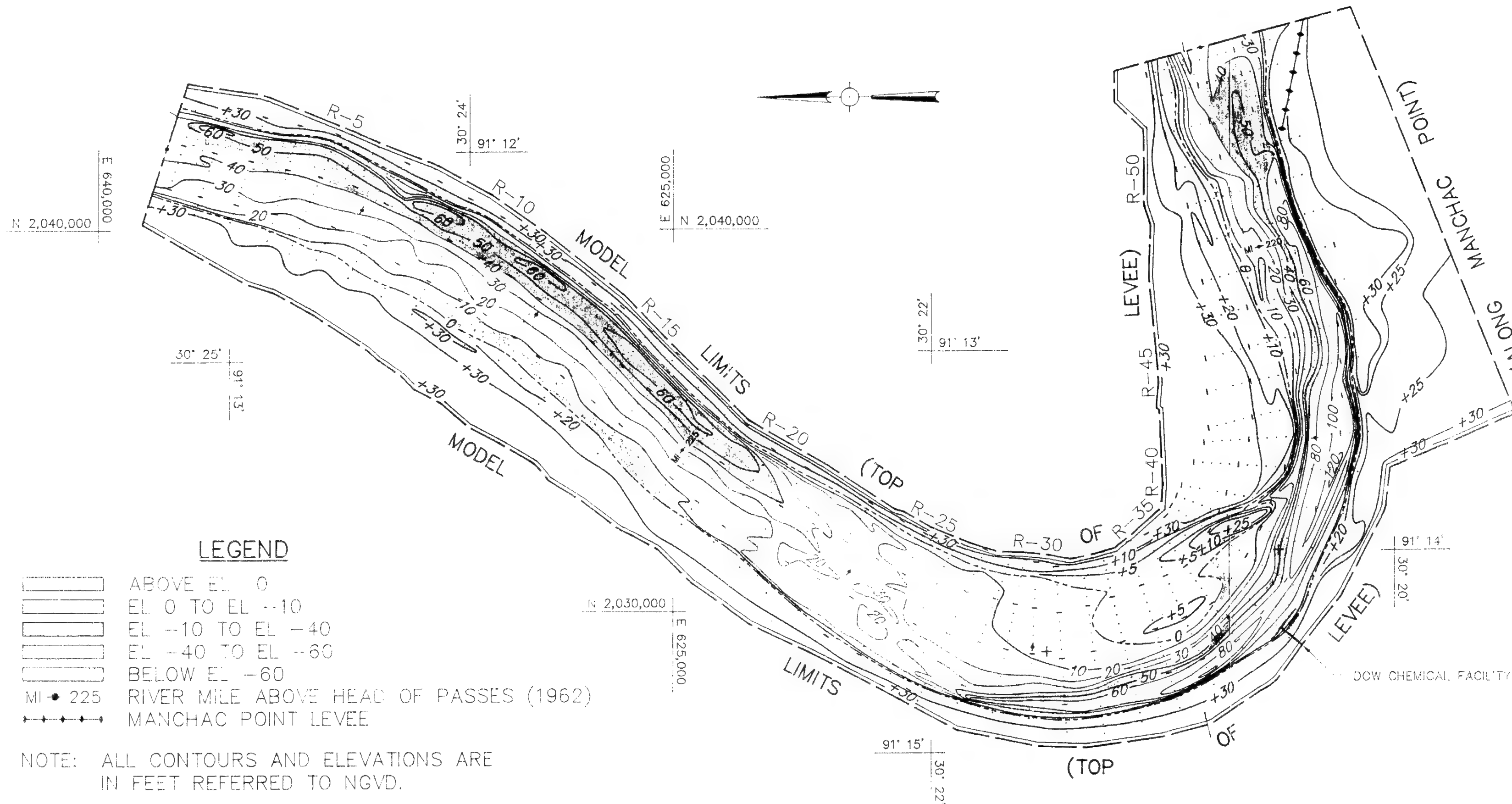


NOTE: GAGE ZERO AT BATON ROUGE, LA IS ZERO FEET NGVD.

2 FT NGVD = 0 LWRP

MODEL STUDY OF
REDEYE CROSSING REACH, MISSISSIPPI RIVER

SEP 1982 - AUG 1983
PROTOTYPE HYDROGRAPH
AT BATON ROUGE, LA



LEGEND

ABOVE EL 0
EL 0 TO EL -10
EL -10 TO EL -40
EL -40 TO EL -60
BELOW EL -60
MI ● 225 RIVER MILE ABOVE HEAD OF PASSES (1962)
MANCHAC POINT LEVEE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD.

1974 LOW WATER REFERENCE PLANE (LWRP) IS 2 FEET NGVD.

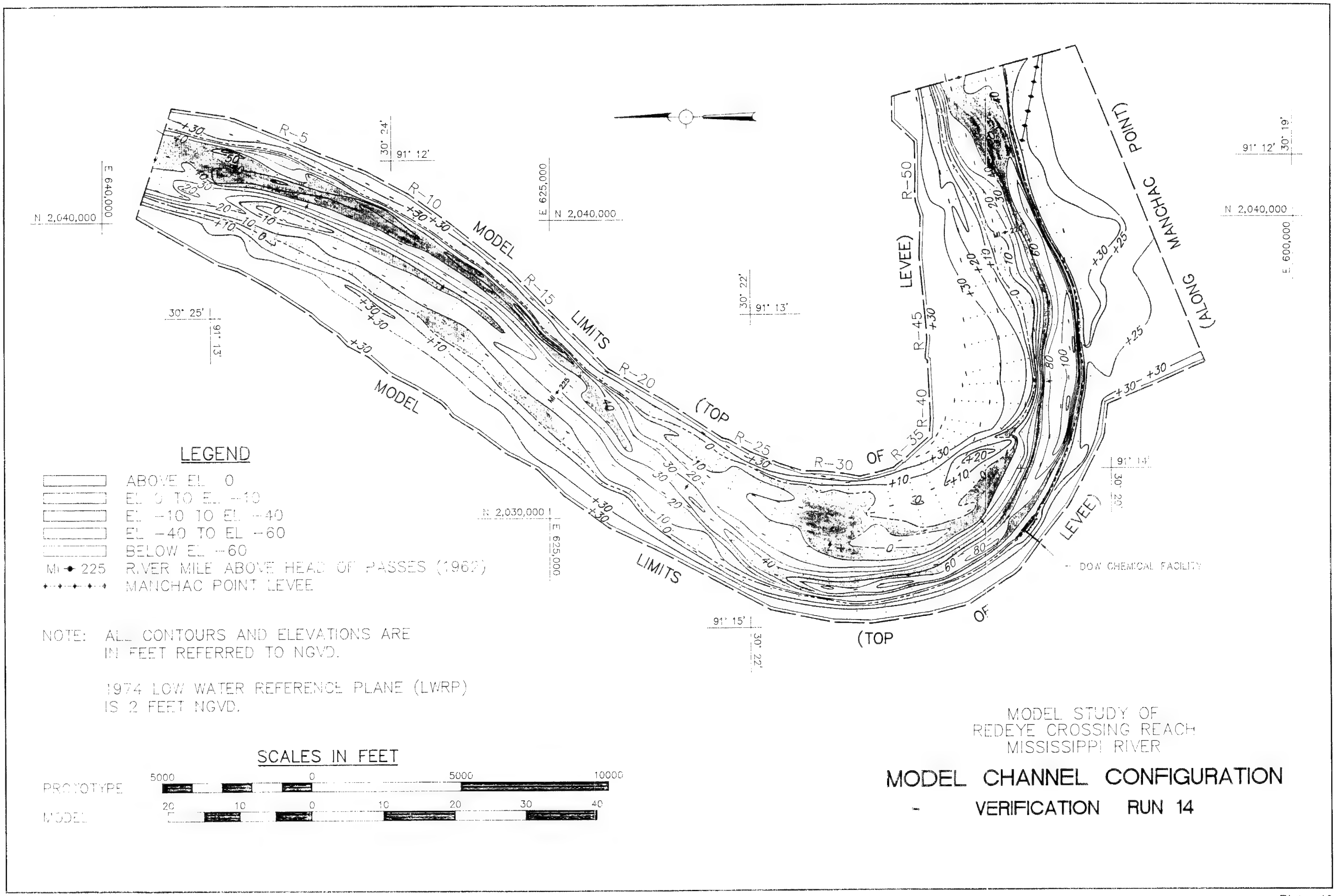
SCALES IN FEET

PROTOTYPE 5000 0 5000 10000

MODEL 20 10 0 10 20 30 40

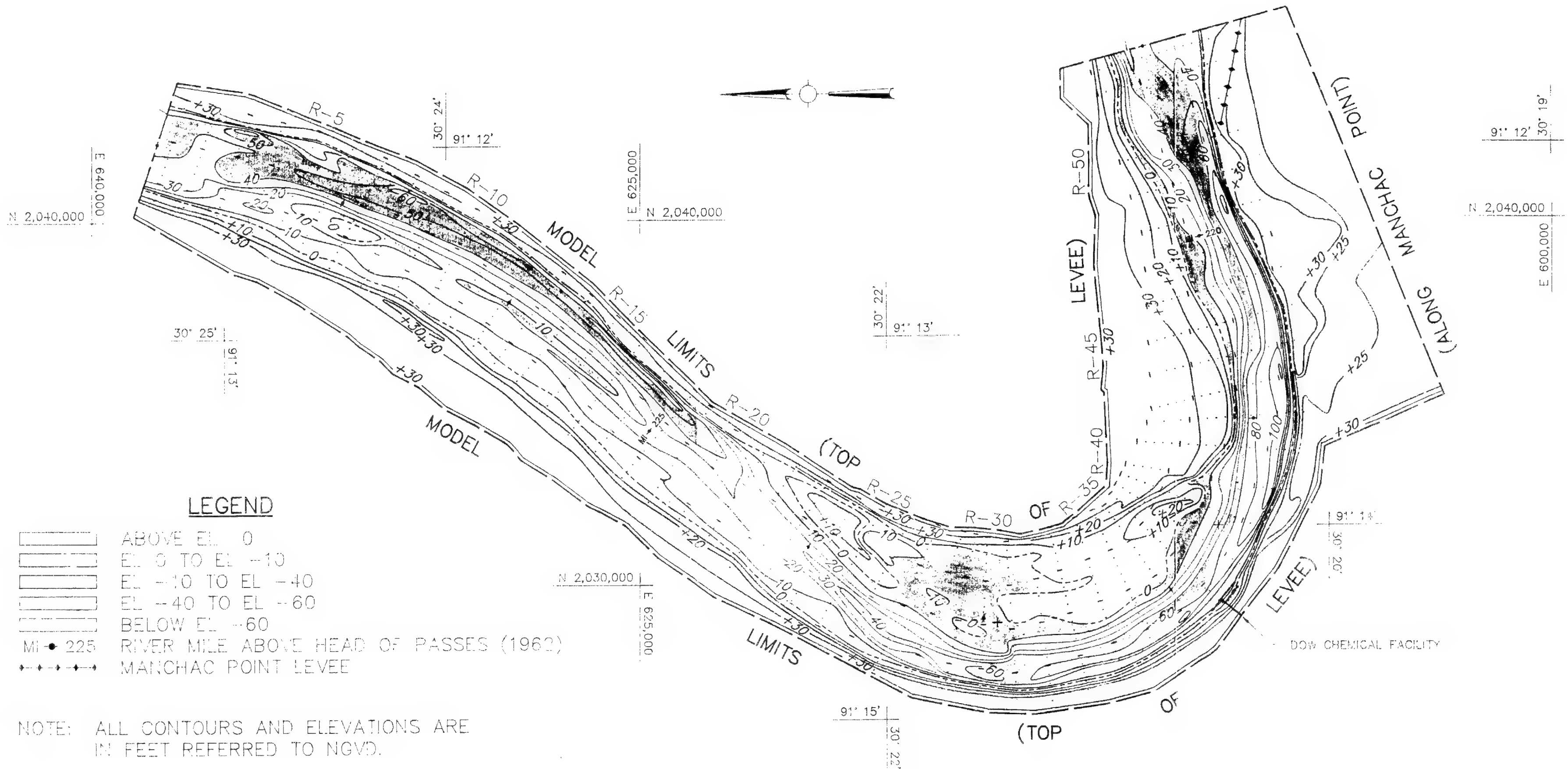
MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER

PROTOTYPE CHANNEL CONFIGURATION
- AUGUST 1983 SURVEY



MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER

MODEL CHANNEL CONFIGURATION
- VERIFICATION RUN 14



LEGEND

- ABOVE EL 0
- EL 0 TO EL -10
- EL -10 TO EL -40
- EL -40 TO EL -60
- BELOW EL -60
- MI • 225 RIVER MILE ABOVE HEAD OF PASSES (1962)
- MANCHAC POINT LEVEE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD.

1974 LOW WATER REFERENCE PLANE (LWRP) IS 2 FEET NGVD

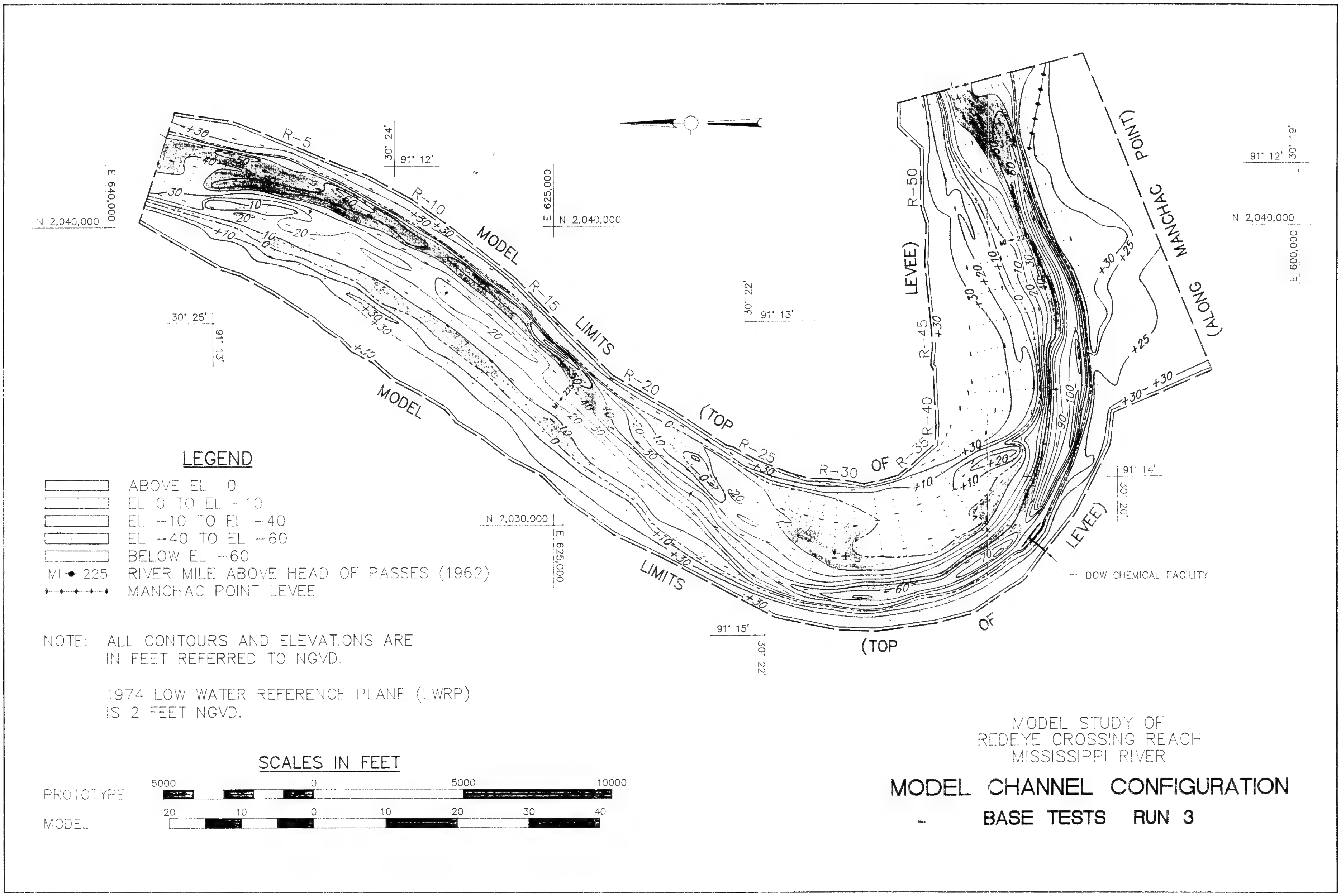
SCALES IN FEET

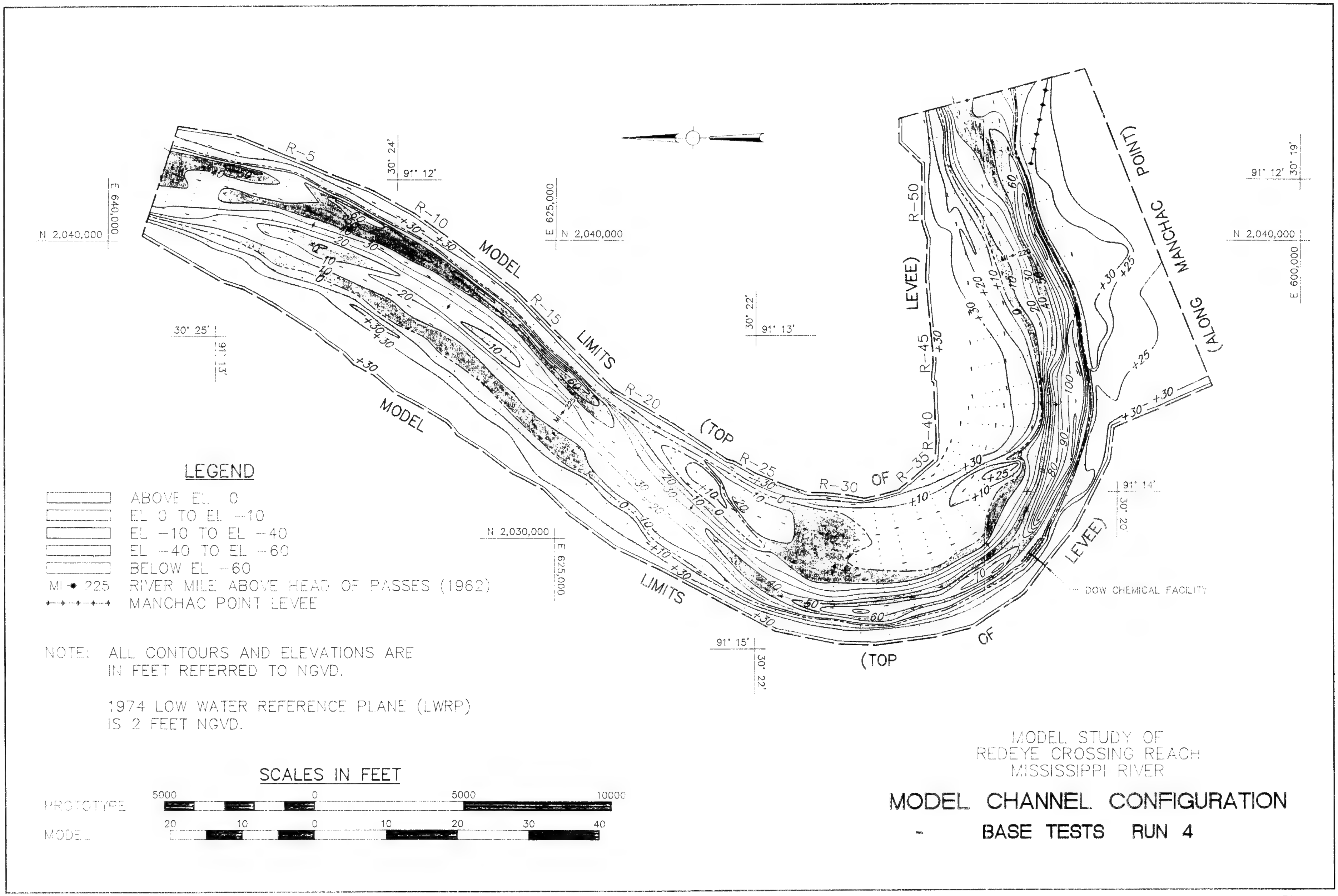
PROTOTYPE 5000 0 5000 10000

MODEL 20 10 0 10 20 30 40

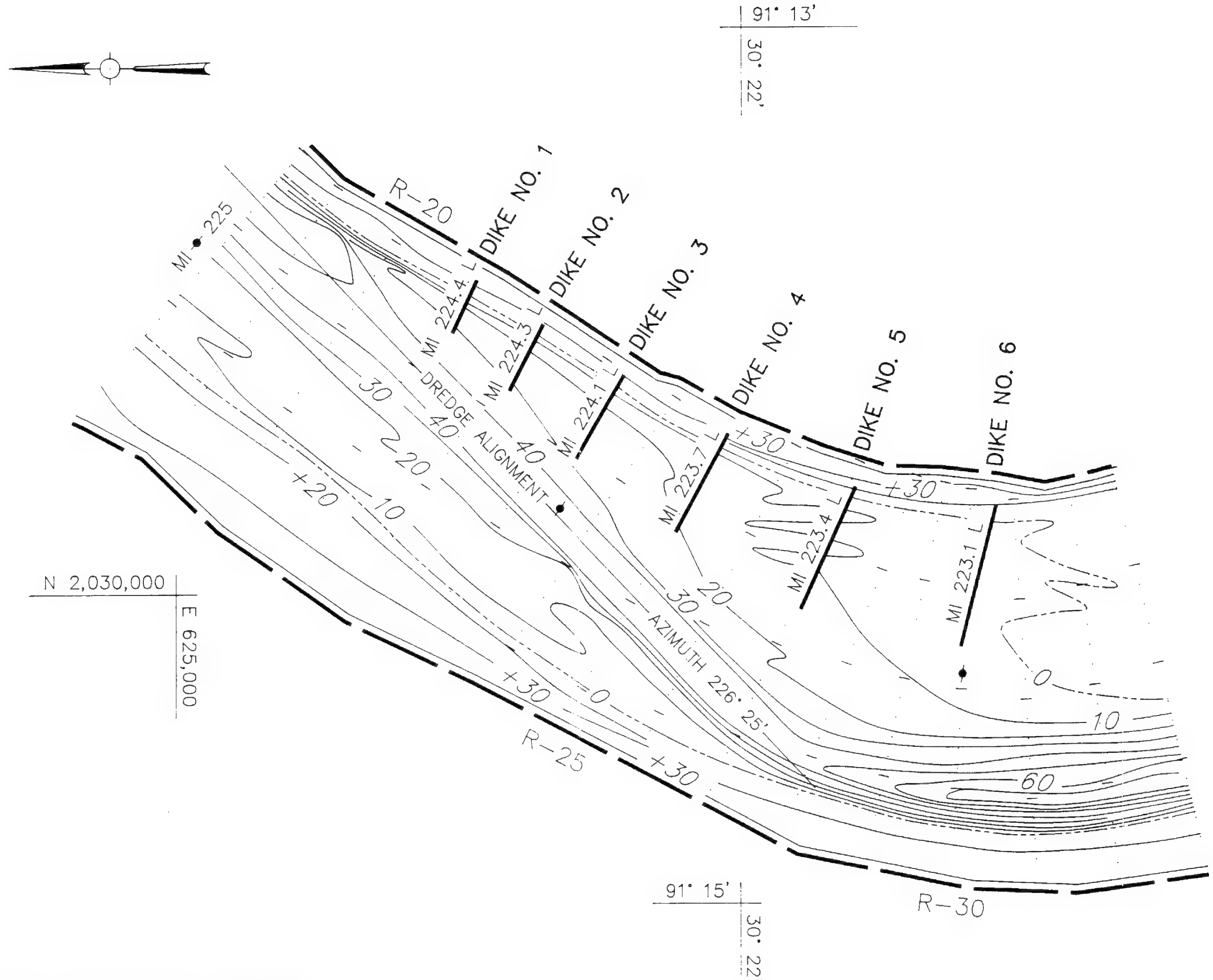
MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER

MODEL CHANNEL CONFIGURATION - VERIFICATION RUN 15


R-5
R-10
R-15
R-20
R-25
R-30
R-35
R-40
R-45
R-50
MODEL LIMITS (TOP OF LEVEE)
(ALONG MANCHAC POINT)
DOW CHEMICAL FACILITY
N 2,040,000
E 640,000
E 625,000
N 2,030,000
E 600,000
30° 24'
91° 12'
30° 25'
91° 13'
30° 22'
91° 15'
91° 14'
30° 20'
30° 19'
LEGEND
ABOVE EL 0
EL 0 TO EL -10
EL -10 TO EL -40
EL -40 TO EL -60
BELOW EL -60
MI ● 225 RIVER MILE ABOVE HEAD OF PASSES (1962)
MANCHAC POINT LEVEE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD.
1974 LOW WATER REFERENCE PLANE (LWRP) IS 2 FEET NGVD.
SCALES IN FEET
PROTOTYPE 5000 0 5000 10000
MODEL 20 10 0 10 20 30 40
MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER
MODEL CHANNEL CONFIGURATION
BASE TESTS RUN 3



R-5
R-10
R-15
R-20
R-25
R-30
R-35
R-40
R-45
R-50
MODEL LIMITS (TOP OF LEVEE)
MODEL LIMITS (TOP OF LEVEE)
(ALONG MANCHAC POINT)
DOW CHEMICAL FACILITY
N 2,040,000
E 640,000
E 625,000
N 2,030,000
E 600,000
30° 24'
91° 12'
30° 25'
91° 13'
30° 22'
91° 15'
91° 14'
30° 20'
30° 19'
LEGEND
ABOVE EL 0
EL 0 TO EL -10
EL -10 TO EL -40
EL -40 TO EL -60
BELOW EL -60
MI • 225 RIVER MILE ABOVE HEAD OF PASSES (1962)
MANCHAC POINT LEVEE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD.
1974 LOW WATER REFERENCE PLANE (LWRP) IS 2 FEET NGVD.
SCALES IN FEET
PROTOTYPE 5000 0 5000 10000
MODEL 20 10 0 10 20 30 40
MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER
MODEL CHANNEL CONFIGURATION
BASE TESTS RUN 4

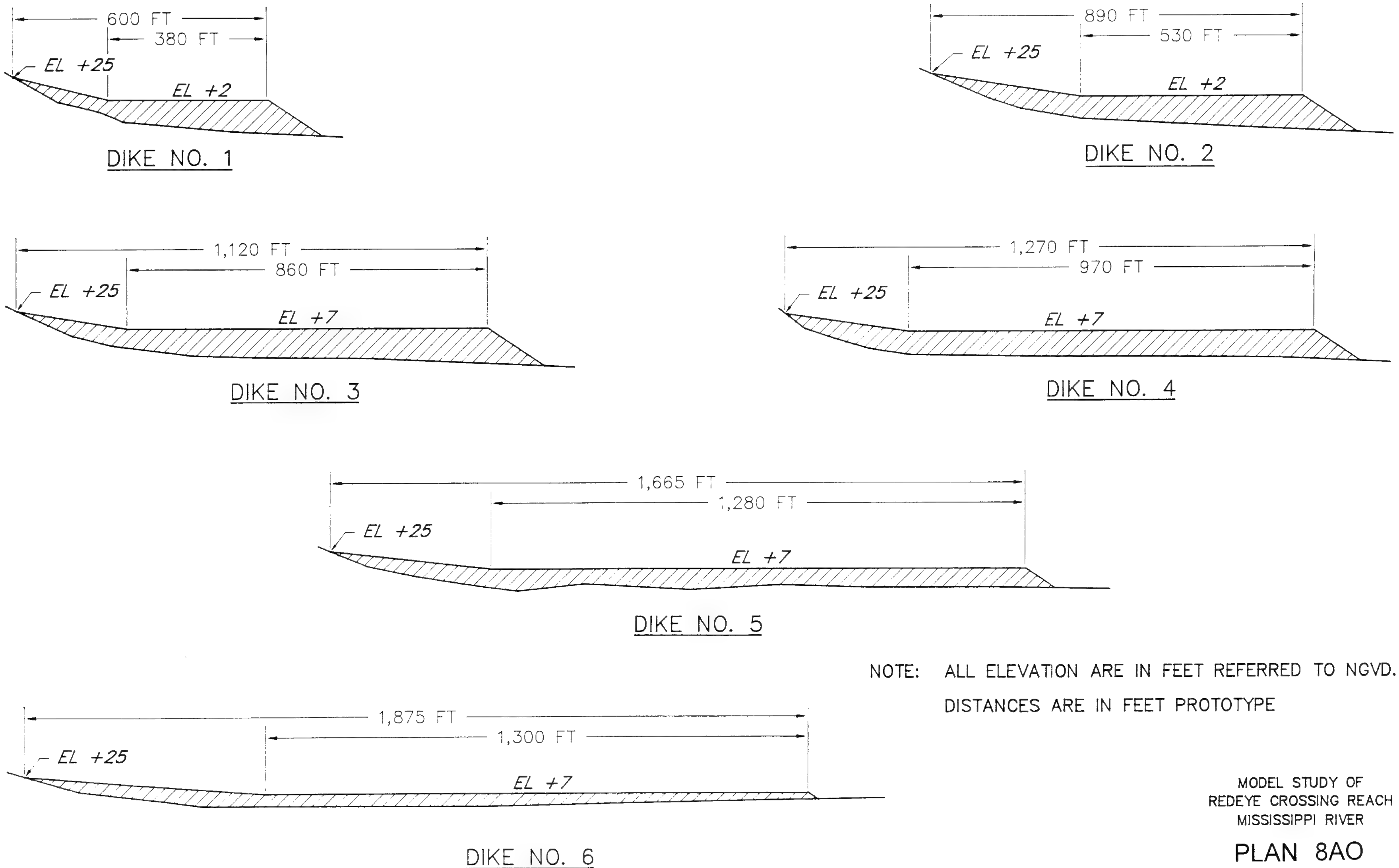


NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD.

1974 LOW WATER REFERENCE PLANE (LWRP) IS 2 FEET NGVD.

MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER

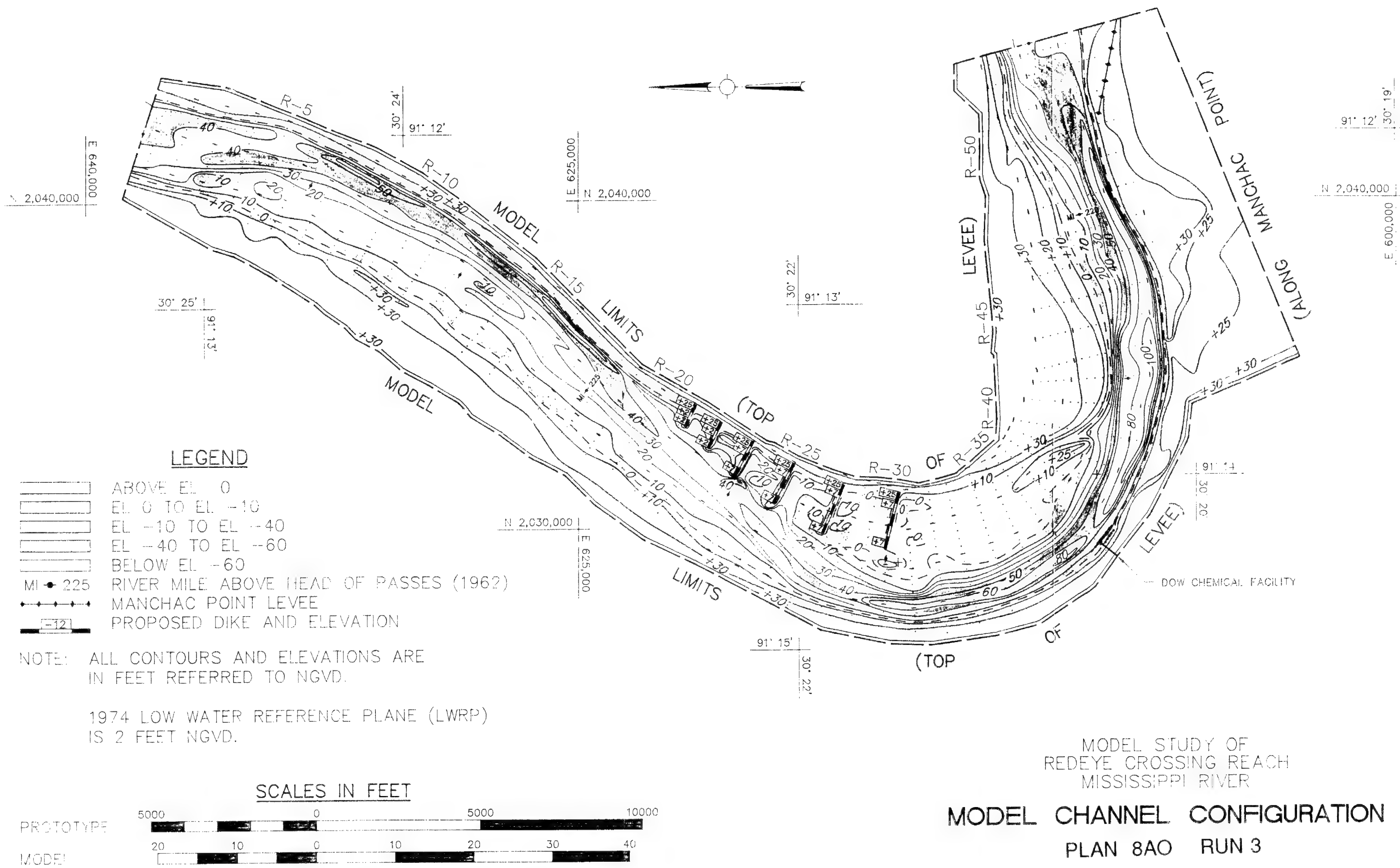
PLAN 8AO



NOTE: ALL ELEVATION ARE IN FEET REFERRED TO NGVD.
DISTANCES ARE IN FEET PROTOTYPE

MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER

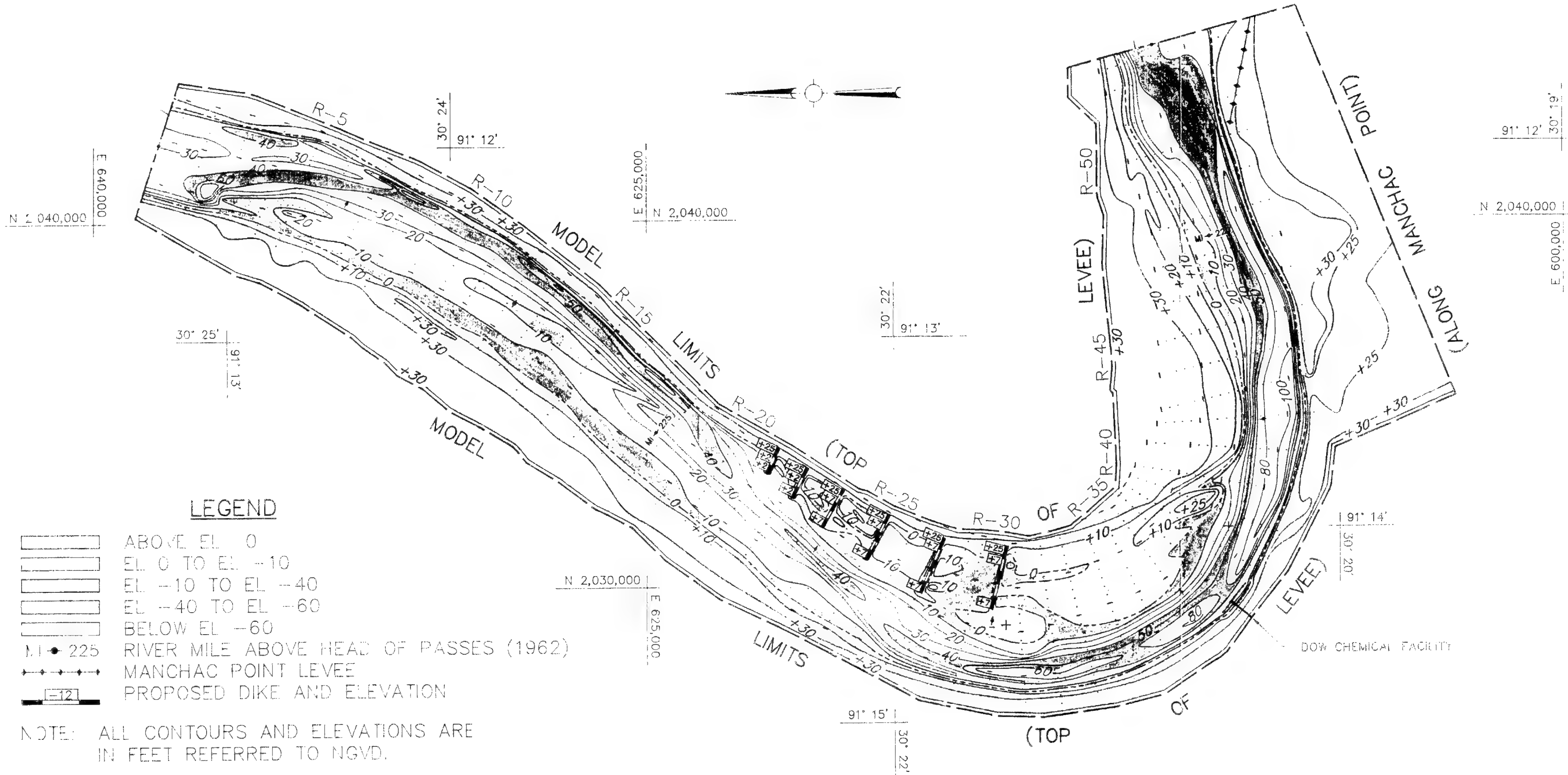
PLAN 8AO
DIKE PROFILES



MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER

MODEL CHANNEL CONFIGURATION

PLAN 8AO RUN 3



LEGEND

- ABOVE EL 0
- EL 0 TO EL −10
- EL −10 TO EL −40
- EL −40 TO EL −60
- BELOW EL −60
- M 225 — RIVER MILE ABOVE HEAD OF PASSES (1962)
- MANCHAC POINT LEVEE
- −12 — PROPOSED DIKE AND ELEVATION

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD.

1974 LOW WATER REFERENCE PLANE (LWRP) IS 2 FEET NGVD.

SCALES IN FEET

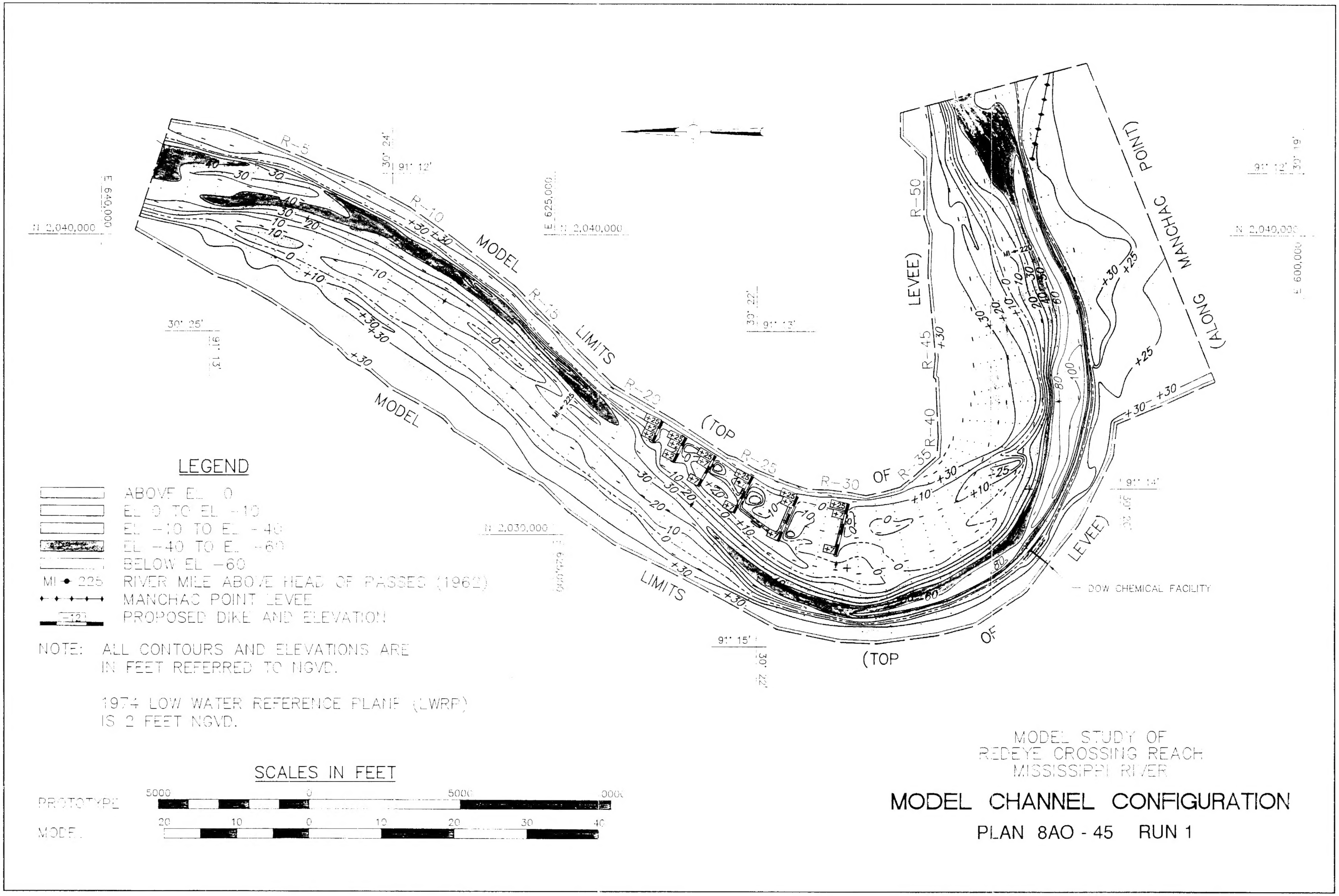
PROTOTYPE 5000 0 5000 10000

MODEL 20 10 0 10 20 30 40

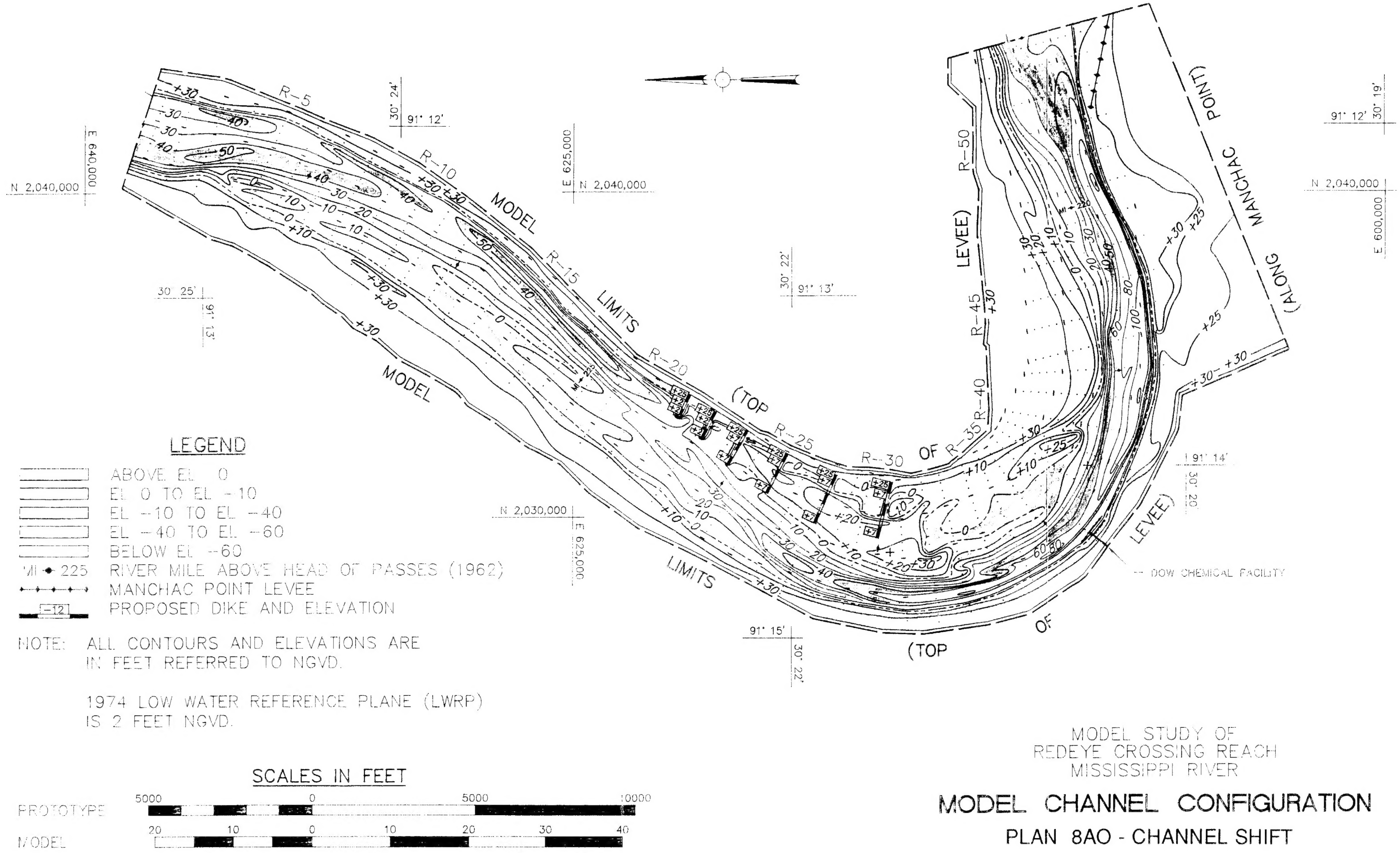
MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER

MODEL CHANNEL CONFIGURATION

PLAN 8AO RUN 4



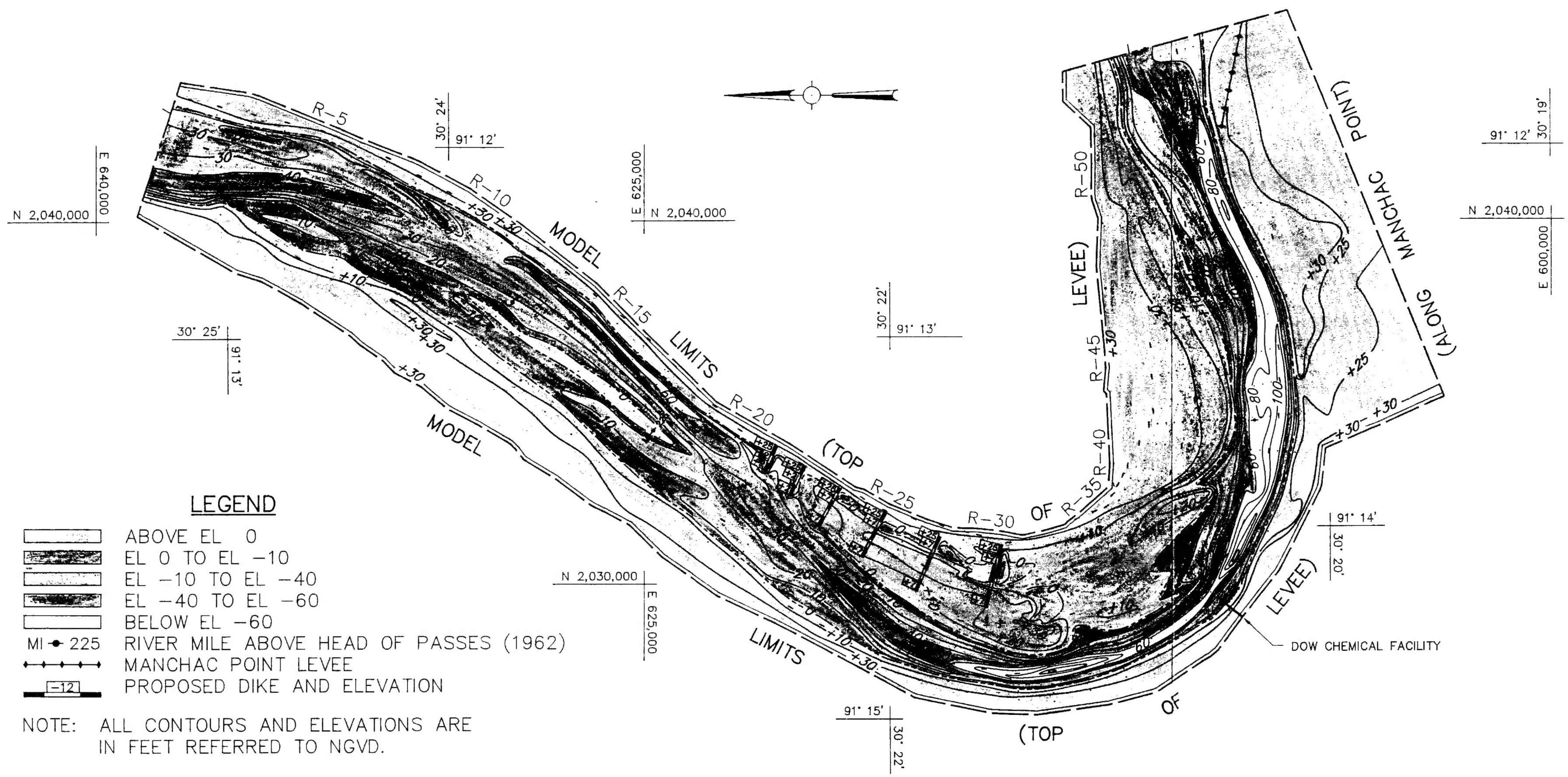
MODEL LIMITS
(TOP OF LEVEE)
(ALONG MANCHAC POINT)
DOW CHEMICAL FACILITY
LEGEND
ABOVE EL 0
EL 0 TO EL -10
EL -10 TO EL -40
EL -40 TO EL -60
BELOW EL -60
MI 225 RIVER MILE ABOVE HEAD OF PASSES (1962)
MANCHAC POINT LEVEE
PROPOSED DIKE AND ELEVATION
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD.
1974 LOW WATER REFERENCE PLANE (LWRP) IS 2 FEET NGVD.
SCALES IN FEET
PROTOTYPE
MODEL
MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER
MODEL CHANNEL CONFIGURATION
PLAN 8AO - 45 RUN 1



MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER

MODEL CHANNEL CONFIGURATION

PLAN 8AO - CHANNEL SHIFT



LEGEND

- ABOVE EL 0
- EL 0 TO EL −10
- EL −10 TO EL −40
- EL −40 TO EL −60
- BELOW EL −60
- MI 225 RIVER MILE ABOVE HEAD OF PASSES (1962)
- MANCHAC POINT LEVEE
- −12 PROPOSED DIKE AND ELEVATION

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD.

1974 LOW WATER REFERENCE PLANE (LWRP) IS 2 FEET NGVD.

SCALES IN FEET

PROTOTYPE 5000 0 5000 10000

MODEL 20 10 0 10 20 30 40

MODEL STUDY OF
REDEYE CROSSING REACH
MISSISSIPPI RIVER

MODEL CHANNEL CONFIGURATION
- PLAN 8 A 1 RUN 1

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

Public reporting burden for this collection of information is estimated to average 1 hour per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestions for reducing this burden, to Washington Headquarters Services, Directorate for Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302, and to the Office of Management and Budget, Paperwork Reduction Project (0704-0188), Washington, DC 20503.

1. AGENCY USE ONLY *(Leave blank)*	2. REPORT DATE	3. REPORT TYPE AND DATES COVERED
	October 1995	Final report

4. TITLE AND SUBTITLE

Redeye Crossing Reach, Lower Mississippi River;
Report 1, Sediment Investigation

5. FUNDING NUMBERS

6. AUTHOR(S)

T. J. Pokrefke, Jr., C. R. Nickles
N. K. Raphelt, M. J. Trawle, M. B. Boyd

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)

U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER

Technical Report
HL-95-13

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)

U.S. Army Engineer District, New Orleans
P.O. Box 60267
New Orleans, LA 70160-0267

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES

Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT

Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

The sedimentation study conducted on the Redeye Crossing Reach of the Mississippi River about 3 miles downstream of Baton Rouge, LA, was a combination of numerical and physical movable-bed model studies to aid in the development of a satisfactory dike design for this reach. A two-dimensional numerical model, TABS-2, was used to predict the reduction in dredging that could be anticipated with the original dike design and subsequent modifications. Those modifications included changing the length, height, location, and number of spur dikes. The plans investigated addressed the required dike plan to maintain the existing 40-ft navigation channel through the reach and an enhancement of that plan to provide a 45-ft channel to be developed in the near future.

Since no dikes presently exist in this portion of the Mississippi River, the physical movable-bed model study was also conducted to take advantage of the capabilities of both types of models. Thus the overall study allowed use of the numerical model to screen plans and the physical model to address detailed impacts of the plans. The physical model was constructed to a horizontal scale of 1:240 and a vertical scale of 1:200 including the river channel and overbank areas to the adjacent levees. During the overall testing program the numerical model was used to refine and test dikes plans. The dike plans deemed most successful from the numerical sedimentation model were also tested on the physical model.

14. SUBJECT TERMS

Mississippi River	Physical model
Navigation channel	Reduce dredging
Numerical model	Sedimentation

15. NUMBER OF PAGES

78

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED		

NSN 7540-01-280-5500

Standard Form 298 (Rev. 2-89)
Prescribed by ANSI Std. Z39-18
298-102